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NUCLEAR ENERGY AGENCY
COMMITTEE ON THE SAFETY OF NUCLEAR INSTALLATIONS

NEA/CSNI/R(2001)13/VOL4
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Workshop on the Seismic Re-evaluation of all Nuclear Facilities

Workshop Proceedings

Ispra, Italy
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English - Or. English

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ORGANISATION FOR ECONOMIC CO-OPERATION AND DEVELOPMENT

Pursuant to Article 1 of the Convention signed in Paris on 14th December 1960, and which came into force on 30th September 1961, the Organisation for Economic Co-operation and Development (OECD) shall promote policies designed:

- to achieve the highest sustainable economic growth and employment and a rising standard of living in Member countries, while maintaining financial stability, and thus to contribute to the development of the world economy;
- to contribute to sound economic expansion in Member as well as non-member countries in the process of economic development; and
- to contribute to the expansion of world trade on a multilateral, non-discriminatory basis in accordance with international obligations.

The original Member countries of the OECD are Austria, Belgium, Canada, Denmark, France, Germany, Greece, Iceland, Ireland, Italy, Luxembourg, the Netherlands, Norway, Portugal, Spain, Sweden, Switzerland, Turkey, the United Kingdom and the United States. The following countries became Members subsequently through accession at the dates indicated hereafter: Japan (28th April 1964), Finland (28th January 1969), Australia (7th June 1971), New Zealand (29th May 1973), Mexico (18th May 1994), the Czech Republic (21st December 1995), Hungary (7th May 1996), Poland (22nd November 1996), Korea (12th December 1996) and the Slovak Republic (14th December 2000). The Commission of the European Communities takes part in the work of the OECD (Article 13 of the OECD Convention).

NUCLEAR ENERGY AGENCY

The OECD Nuclear Energy Agency (NEA) was established on 1st February 1958 under the name of the OEEC European Nuclear Energy Agency. It received its present designation on 20th April 1972, when Japan became its first non-European full Member. NEA membership today consists of 27 OECD Member countries: Australia, Austria, Belgium, Canada, Czech Republic, Denmark, Finland, France, Germany, Greece, Hungary, Iceland, Ireland, Italy, Japan, Luxembourg, Mexico, the Netherlands, Norway, Portugal, Republic of Korea, Spain, Sweden, Switzerland, Turkey, the United Kingdom and the United States. The Commission of the European Communities also takes part in the work of the Agency.

The mission of the NEA is:

- to assist its Member countries in maintaining and further developing, through international co-operation, the scientific, technological and legal bases required for a safe, environmentally friendly and economical use of nuclear energy for peaceful purposes, as well as
- to provide authoritative assessments and to forge common understandings on key issues, as input to government decisions on nuclear energy policy and to broader OECD policy analyses in areas such as energy and sustainable development.

Specific areas of competence of the NEA include safety and regulation of nuclear activities, radioactive waste management, radiological protection, nuclear science, economic and technical analyses of the nuclear fuel cycle, nuclear law and liability, and public information. The NEA Data Bank provides nuclear data and computer program services for participating countries.

In these and related tasks, the NEA works in close collaboration with the International Atomic Energy Agency in Vienna, with which it has a Co-operation Agreement, as well as with other international organisations in the nuclear field.

COMMITTEE ON THE SAFETY OF NUCLEAR INSTALLATIONS

The NEA Committee on the Safety of Nuclear Installations (CSNI) is an international committee made up of scientists and engineers. It was set up in 1973 to develop and co-ordinate the activities of the Nuclear Energy Agency concerning the technical aspects of the design, construction and operation of nuclear installations insofar as they affect the safety of such installations. The Committee's purpose is to foster international co-operation in nuclear safety amongst the OECD Member countries.

CSNI constitutes a forum for the exchange of technical information and for collaboration between organisations which can contribute, from their respective backgrounds in research, development, engineering or regulation, to these activities and to the definition of its programme of work. It also reviews the state of knowledge on selected topics of nuclear safety technology and safety assessment, including operating experience. It initiates and conducts programmes identified by these reviews and assessments in order to overcome discrepancies, develop improvements and reach international consensus in different projects and International Standard Problems, and assists in the feedback of the results to participating organisations. Full use is also made of traditional methods of co-operation, such as information exchanges, establishment of working groups and organisation of conferences and specialist meeting.

The greater part of CSNI's current programme of work is concerned with safety technology of water reactors. The principal areas covered are operating experience and the human factor, reactor coolant system behaviour, various aspects of reactor component integrity, the phenomenology of radioactive releases in reactor accidents and their confinement, containment performance, risk assessment and severe accidents. The Committee also studies the safety of the fuel cycle, conducts periodic surveys of reactor safety research programmes and operates an international mechanism for exchanging reports on nuclear power plant incidents.

In implementing its programme, CSNI establishes co-operative mechanisms with NEA's Committee on Nuclear Regulatory Activities (CNRA), responsible for the activities of the Agency concerning the regulation, licensing and inspection of nuclear installations with regard to safety. It also co-operates with NEA's Committee on Radiation Protection and Public Health and NEA's Radioactive Waste Management Committee on matters of common interest.

FOREWORD

The Committee on the Safety of Nuclear Installations (CSNI) of the OECD-NEA co-ordinates the NEA activities concerning the technical aspects of design, construction and operation of nuclear installations insofar as they affect the safety of such installations.
The Integrity and Ageing Working Group (IAGE WG) of the CSNI deals with the integrity of structures and components, and has three sub-groups, dealing with the integrity of metal components and structures, ageing of concrete structures, and the seismic behaviour of structures. This workshop was proposed by the sub-group dealing with the seismic behaviour of structures.

Seismic re-evaluation is identified as the process of carrying out a re-assessment of the safety of existing nuclear facilities for a specified seismic hazard. This may be necessary when no seismic hazard was considered in the original design of the plant, the relevant codes and regulations have been revised, the seismic hazard for the site has been re-assessed or there is a need to assess the capacity of the plant for severe accident conditions and behaviour beyond the design basis. Re-evaluation may also be necessary to resolve an issue, or to assess the impact of new findings or knowledge.

In 1997, CSNI recognised the increasing importance of seismic re-evaluation for nuclear facilities throughout the world. It prepared a status report on seismic Re-evaluation NEA/CSNI/R(98)5 which summarized the current situation for Member countries of the OECD. The report suggested a number of areas of the seismic reevaluation process, which could be considered in the future. In May 2000, the seismic sub-group reviewed these suggestions and determined that it was timely to address progress on this topic through this workshop. The workshop focused on methods and acceptance criteria and, on countermeasures and strengthening of plant.

The workshop had 2 technical sessions listed below devoted to presentations, and a 3rd session devoted to a discussion of the material presented and to the formulation of workshop conclusions to update conclusions of the 1998 report.

Session 1

- Methods and acceptance criteria
- Benefits and disadvantages of the various methods of re-evaluation (Seismic PSA, Margins, deterministic, databases, tests …) in particular circumstances
- Role and scope of the peer review process
- Definition of the scope of the plant to be selected for the re-evaluation process
- Differences between re-evaluation and design criteria

Session 2

- Countermeasures/strengthening
- Civil engineering structures
- Post earthquake procedures and measures
- Strategies and priorities
- Recent innovation or research outputs

In the area of the seismic behaviour of structures, the CSNI is currently preparing among others a workshop on relations between seismological data and seismic engineering analysis to evaluate uncertainties and margins through a better description of real ground motion spectrum as opposed to a ground response design. Short reports on "lessons learned from high magnitudes earthquakes with respect to nuclear codes and standards" are under preparation and will cover several recent earthquakes.

Seismic reports issued by the group since 1996 are:

- NEA/CSNI/R(1996)10 Seismic shear wall ISP: NUPEC's seismic ultimate dynamic response test: comparison report, 1996. also referenced as: OCDE/GD(96)188

- NEA/CSNI/R(1996)11 Report of the task group on the seismic behaviour of structures: status report, 1997. also referenced as: OCDE/GD(96)189

- NEA/CSNI/R(1998)5 Status report on seismic re-evaluation, 1998.

- NEA/CSNI/R(1999)28 Proceedings of the OECD/NEA Workshop on Seismic Risk, CSNI PWG3 and PWG5, Tokyo, Japan 10-12 August 1999.

- NEA/CSNI/R(2000)2/VOL1 Proceedings of the OECD/NEA Workshop on the "Engineering Characterisation of Seismic Input, BNL, USA 15-17 November 1999 -

- NEA/CSNI/R(2000)2/VOL2 Proceedings of the OECD/NEA Workshop on the "Engineering Characterisation of Seismic Input, BNL, USA 15-17 November 1999

The complete list of CSNI reports, and the text of reports from 1993 onwards, is available on http://www.nea.fr/html/nsd/docs/

Acknowledgement

Gratitude is expressed to the European Commission Joint Research Centre, Ispra (VA), Italy for hosting the workshop as well as to the Organization for Economic Co-operation and Development (OECD) / Nuclear Energy Agency (NEA) / Committee on the Safety of Nuclear Installations (CSNI) / Integrity and Aging Working Group (IAGE) (Integrity of Components and Structures) for sponsoring our work.
Thanks are also expressed to chairmen of the sessions for their effort and co-operation.

The organizing Committee members were:

Mr John Donald, HSE (UK)
Mr Jean-Dominique Renard, Tractebel (B)
Dr Vito Renda, JRC/ISPRA (I)
Prof. Pierre Labbé, IAEA
Dr. Tamas Katona, PAKS (HU)
Dr Andrew Murphy, USNRC (USA)
Mr Eric Mathet, OECD/NEA

OECD/NEA WORKSHOP ON THE SEISMIC RE-EVALUATION OF ALL NUCLEAR FACILITIES

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OECD-NEA Workshop on the Seismic Re-evaluation of all Nuclear Facilities,
EC JRC/Ispra, Italy

PROBABILISTIC SEISMIC ANALYSIS OF SAFETY RELATED STRUCTURES OF KOZLODUY NPP

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Risk Engineering Ltd., Sofia, Bulgaria

1. Introduction

The paper presents results achieved within the PSA projects on units of Kolzoduy NPP. The NPP consists of 6 units, all of them of VVER type (2 Units are of 1000MW and 4 units are 440/230 type). The PSA of the 1000 MW units was performed in 1993-1994. The results were used for triggering of large modernization program that is under implementation. The 440MW units have been analyzed recently in order to assess the effectiveness of the seismic upgrading.

The probability of failure of a structure for an expected lifetime can be obtained from the annual frequency of failure, $\mathcal{E}_F$, determined by the relation [1]:

$$\mathcal{E}_F = \int [d[\mathcal{E}(x)]/dx]\, P(f|x)\, dx$$

$\mathcal{E}(x)$ is the annual frequency of exceedance of load level x (for example, the variable x may be peak ground acceleration), $P(f|x)$ is the conditional probability of structure failure at a given seismic load level x. The problem leads to the assessment of the seismic hazard $\mathcal{E}(x)$ and the fragility $P(f|x)$.

The seismic hazard curves are obtained by the probabilistic seismic hazard analysis. The fragility curves are obtained after the response of the structure is defined probabilistically and its capacity and the associated uncertainties are assessed. Finally the fragility curves are combined with the seismic loading to estimate the frequency of failure for each critical scenario.

2. Probabilistic hazard analyses

The seismicity of Kozloduy region is well defined and studied. The earthquake distribution in the 320 km area around the NPP site is taken into consideration. The earthquake sources could be divided into three groups: local sources (zone with radius of 30 km and expected maximum magnitude Mmax,exp=4.50), shallow depth sources (Mmax,exp=8) and intermediate depth sources from Vrancea region generating long-period excitations (Mmax,exp=7.8). The probabilistic assessment of the seismic hazard is developed on the base of some assumptions [2]: existence of a model of the potential seismogenic zones; the geometry of those zones, the frequency of earthquake realization as well as the maximum magnitude value of generated earthquakes and attenuation law are known. The random and model uncertainties are taken into account. Several alternative assumptions are considered - two attenuation laws for the Vrancea seismic zone and two laws for the shallow depth sources; linear and quadratic regression curves for frequency of occurrence; two variants of maximum expected magnitude and variants of the space distribution of the seismic sources - fault model or diffuse seismicity. The computation is performed for 36 cases of different alternatives for 9 different periods of the response spectrum and damping 5% of the critical. As a result the mean value, standard deviation, median and geometric deviation (log-normal distribution) of the maximum acceleration and the spectral accelerations are determined. Hazard curves for the maximum acceleration at the site of Kozloduy are derived (fig.1). The equal hazard acceleration response spectra for annual probability of exceedance of 0.001, 0.0001 and 0.00001 are computed. Figure 2 illustrates the spectra for the second hazard level.

For response analysis the seismic hazard is presented by a set of modified artificial accelerograms - 10 accelerograms (three components) for each level of annual probability of exceedance. In order to

determine the necessary statistics for generation of the accelerograms 90 pre-selected records of real earthquakes divided in three groups corresponding to the seismic zones are analysed. The respective acceleration response spectra are statistically investigated. That information is needed for the generation of artificial accelerograms. Their three components are statistically independent and their relative intensity corresponds to that of the real accelerograms.

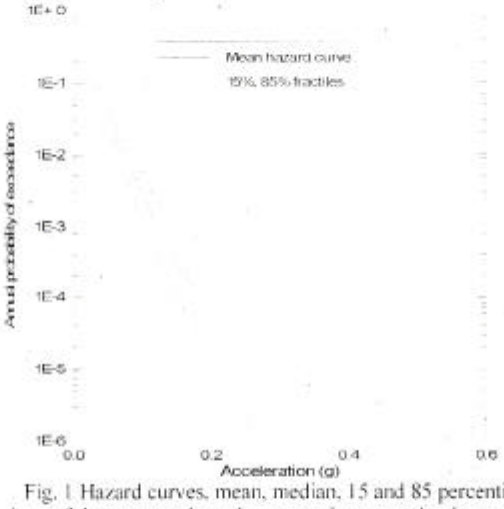

Fig. 1 Hazard curves, mean, median, 15 and 85 percentiles

The time envelope of the generated accelerograms is assumed to be trapezoidal. The total duration varies between 15 and 60 s. The duration of the intensive part and that of the beginning are also varied. Using this information acceleration response spectra are generated. They match the respective equal hazard spectrum (Fig. 3). Then accelerograms are generated and for each hazard level the mean value and the standard deviation of their spectra (10 spectra for each component) are computed. They match very well the mean value and the variation of the uniform hazard spectra.

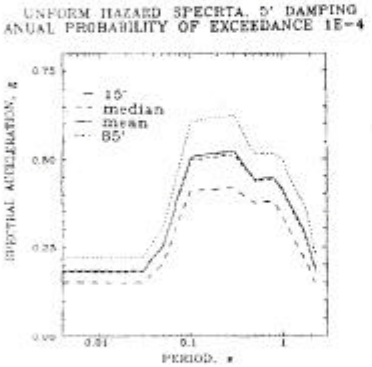

Fig. 2. Acceleration response spectra (free field)

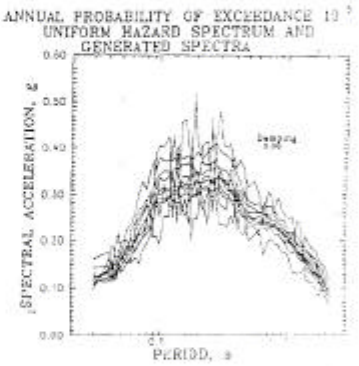

Fig.3. Generated response spectra

All those accelerograms refer to the free field of the site. They are transferred to the foundation level by deconvolution procedure. A probabilistic model of the local geological stratum is compiled consisting of ten geological profiles generated by LHCED [5]. For each level of hazard the mean and mean plus one standard deviation response spectra of the respective accelerograms at foundation level are computed. In such way the input seismic motion for each hazard level is presented by ten three component accelerograms taking into consideration the local soil conditions.

3. Basic Formulation of Fragility Curve Model

Most frequently the objective of the fragility evaluation is to estimate the peak ground motion acceleration value for which the seismic response of a structure (system, component) exceeds the capacity resulting in failure. The estimation of the ground acceleration value could be performed on the base of calculations or based on experience data (the later could be from real earthquakes or dynamic tests). Because there are many sources of variability the structure (component) fragility is expressed usually by family of curves. A probability value is assigned to each curve to reflect the uncertainty in the fragility estimation.

One simple but effective fragility model [3] supposes that the entire family of curves representing a particular failure mode can be expressed by median ground acceleration A_m and two random variables ε_R and ε_U, thus the ground acceleration capacity A is given by:

$$A = A_m \varepsilon_R \varepsilon_U$$

ε_R and ε_U are log-normally distributed with unit medians and standard deviations β_R and β_u respectively. They represent the inherent randomness about the median and the uncertainty of the median value respectively. In some cases the composite variability β_c is used, defined by:

$$\beta_c = (\beta_R^2 + \beta_u^2)^{1/2}$$

The use of β_c and A_m provides a single best estimate fragility curve which does not explicitly separate randomness from uncertainty.

There are two basic possibilities used to estimate the key parameters A and β. The first possibility is to use scaling of already available analyses with known safety factor. The factor of safety F is defined as: F=(Actual seismic capacity)/(Actual response due to DE), where DE is the design earthquake. Further on the factor of safety can be expressed by:

$$F = F_S . F_\beta . F_{RS}$$

Where the partial safety factors are: F_S is called stress factor, F_β is the inelastic energy absorption factor, F_{RS} is the structural response factor.

The median and logarithmic standard deviation of the safety factor F are expressed as:

$$mF = mF_S . mF_\beta . mF_{RS}$$

and

$$\beta_F = (\beta_S^2 + \beta_\beta^2 + \beta_{RS}^2)^{1/2}$$

The logarithmic standard deviation could be further divided into random variability and uncertainty. The median factor of safety multiplies the design ground acceleration to obtain the median ground capacity.

The second approach in assessing the fragility is based on generations and direct assessment of the conditional probability of failure. The conditional probability of failure is computed by the expression [4]:

$$P_f = \int FR(x) fL(x) dx$$

where FR(x) is the distribution function of the resistance and fL(x) is the density function of the seismic loading distribution. The direct assessment of the conditional probability of failure should be performed for several load levels, i.e. several points (as minimum 2) of the fragility curve have to be determined. The curve could be found by least square method or other interpolation technique.

4. Case studies

Below results from three cases anlyzed are presented where the PSA technique is used for different purposes [6,7]. In the first case the seismic PSA is used to demonstrate the available safety margin. In the

second case the effectiveness of proposed seismic upgrading is analyzed, the third case is showing analysis used for the purpose of prioritization.

4.1. Case 1. VVER-1000 seismic analysis

The structure of the reactor building of Unit 5 is a complicated spatial stmctural system consisting of four main parts - foundation block, containment shell, auxiliary building and inner reinforced concrete structure. Those four parts are joined by a thick reinforced concrete plate at level 13.2 m.

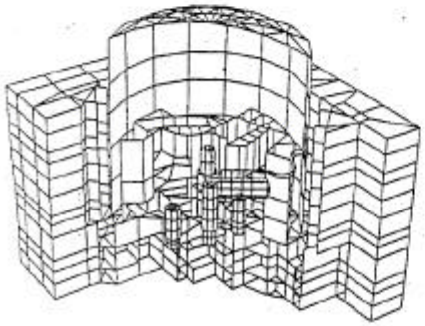

Fig. 4. Soil-strycture-equipment model

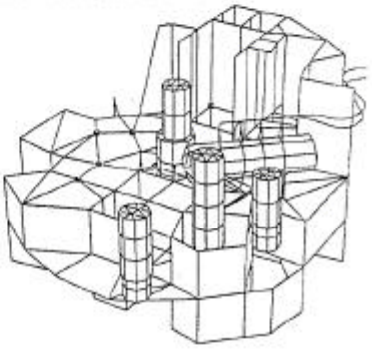

Fig. 5. Equipment model (cross section)

The foundation block consists of a thick reinforced concrete plate with thickness of 2.40 m at elevation 7.00 m and two other plates with thickness of 0.60 m at elevations 0.00 m and 6.60 m. The plates are connected by numerous thick concrete walls. The containment shell is constructed by prestressed concrete. The auxiliary building is composed by reinforced concrete walls and plates situated around the shell and elevated above level 13.20 m. The inner concrete structure consists of many concrete walls and plates. It is situated in the containment shell. The main equipment is anchored in those elements.

A 3D finite element model for dynamic investigation of the soil-structure-equipment system is developed. A proper modeling of all bearing elements is aimed, taking into account all possible structural interactions. Because of the nature of the structure special attention is paid to the modeling of the separation joint between the auxiliary building and the containment shell. The foundation structure is extremely rigid. It is modeled by 1430 plate elements. The containment is modeled also by shell elements. The ring support of the semi-spherical dome is modeled by beam elements. The internal concrete stmcture, placed in the containment above the elevation 13.2m is a complicated spatial system of walls and slabs. The modeling of that structure is very important for the equipment analyses. The auxiliary building is mounted also at elevation 13.2m. It surrounds the containment shell.

The soil is represented by springs and dashpots. The spring constants corresponding to the soil stiffness characteristics (six components) and the respective damping characteristics are estimated using different methods - the semi-empirical method of the weightless spring, the method of the elastic half space and the method of the impedance matrix for a stratified half space. The values of the foundation stiffness characteristics computed by the different methods are practically identical.

The modelling and computation of the soil-structure-equipment system is performed in three stages. First stage concerns the elaboration of the 3-D model of the structure only (fixed in the base). In the second stage 3-D model of the soil-structure system is developed adding springs and dashpots to the base mat. Finally the complex spatial model of the soil-structure-equipment system is created. A cross section of the model is shown in Figure 4. Only the main equipment (reactor, steam-generator, pressuriser, emergency cooling water tank, pumps, turbines, pipes, etc) participates in the model. Because of the double symmetry only 1/4 of the primary circuit is included as flexible elements. The influence of the

other equipment is taken into consideration by their masses lumped at different points of the main structure. The model of the main equipment is shown in Figure 5.

The damping in the model is computed according to the composite damping rule. In the structure are used 4%, 5% and 7% of the critical damping for the three hazard levels respectively (50% variation). The radiation damping in vertical direction is assumed to be 60%, 70% and 80% (variation 50%), in horizontal direction - 60% of the vertical, for rocking - 50% and for torsion - 30% of the vertical damping respectively.

The modal analysis is performed using 255 natural modes up to the frequency of 25 Hz. Variation 30% of the natural frequencies is applied (frequency shifting).

All computations are performed according to LHCED procedure [5]. The response is computed for all ten three-components accelerograms involving the above mentioned variations and for three levels of hazard. Then a statistical analysis is made for each hazard level and mean values and standard deviations of responses are determined. Three components of the acceleration response spectra at different places (important for the structure and the equipment) are computed. In Figure 7 are shown the spectra for the first hazard level at the mid point of the dome. For various locations the cumulative log-normal distribution of the maximum acceleration is determined. In Figure 8 is given the cumulative distribution of the acceleration of the three components for hazard level 0.001 at the same nodal point. The maximum displacements of different points of the model are computed also.

The efforts in some of the plane elements of the model (the main bearing structural elements) as well as the efforts in the beam elements (the main piping elements) are statistically treated. The mean and mean plus one standard deviation values of axial and shear internal forces and of bending and torque moments are determined.

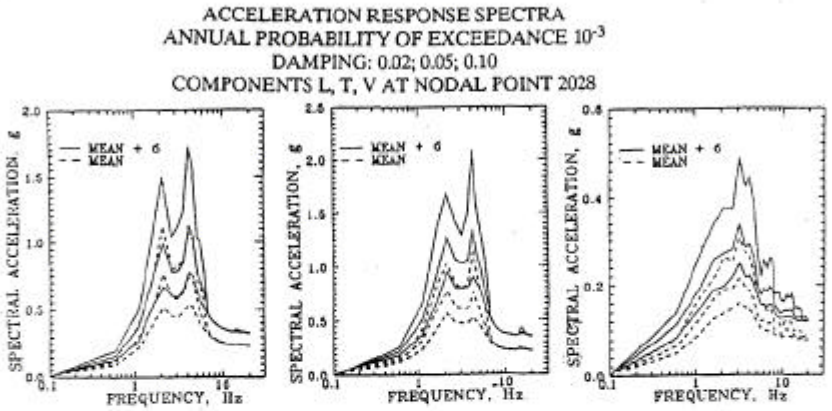

Fig. 6. Probabilistic response spectra at the middle of the domel

The fragility curves are obtained taking into account the design strength characteristics of structural materials. The analytical procedure of Newmark for determining the acceleration at failure is applied. The distribution of this acceleration is obtained assuming log-normal distribution of the random variables. For the most important elements of the structure and equipment the following parameters are computed: the median value of the respective fragility parameter, the random uncertainty Vr, the model uncertainty Vu and the HCLPF (High Confidence Low Probability Failure).

Several scenarios for containment failure are considered in a fragility analysis. The fragility curves are developed according to the methodology described in section 3. Hereafter the fragility

parameters are presented only for 3 scenarios. The first one is shear failure of containment at level 13.20. The second scenario is tension failure due to bending of containment at level 13.20. The third one is impact between auxiliary building and containment at level 45.60. The last scenario is relatively rear because containment structures are usually stand alone structure. The VVER1000 layout is different, i.e containment structure is surrounded by an auxiliary structure. The third scenario is examining the possibility of impact between the containment and the surrounding auxiliary structure. The characteristic values of the fragility curves are presented in table 1. The fragility curves for the case of impact are plotted in figure 8.

CUMULATIVE DISTRIBUTION OF PEAK ACCELERATION
ANNUAL PROBABILITY OF EXCEEDANCE 10^{-4}
(a,b,c) L, T, V COMPONENTS AT NODAL POINT 2028

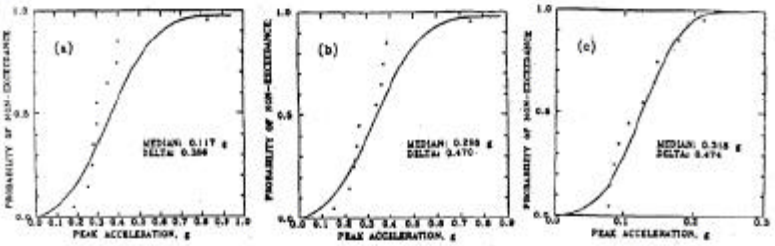

Fig. 7. Cumulative distribution of generated peak acceleration at the midpoint of dome

Table 1. Fragility parameters of Unit 5, Kozloduy NPP containment

Scenario	A_F	HCLPF	u^I	u^{II}
Shear failure at the base, level +13.20m	4.50	3.01	0.06	0.18
Tension bending failure at the base, level +13.20m	8.14	5.39	0.1	0.15
Impact between auxiliary .building and containment, level +45.60m	2.49	1.46	0.1	0.225

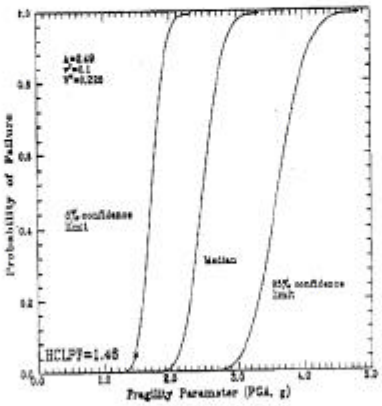

Fig.8 Fragility curves – Impact between auxiliary building and containment at level 45.60

4.2. Case 2. Turbine building seismic upgrading

The steel frame turbine building of VVER 1000 is analyzed. The building is not safety relevant, but is important for electricity production. The outcomes of the probabilistic analysis have been used in a seismic upgrading project as measure for the upgrade effectiveness.

The structure is a 2 bay steel frame. The first span is 45m and with a height of 38m and the second span is 12 m with a height 42m. In longitudinal direction the steel columns are placed in 3 rows (A, B, C) and 12 transversal axes (12m between two adjacent axes). They are connected by longitudinal hinged beams. Diagonal bracing provides the longitudinal stiffness. The foundations are single footings. The roof structure is steel truss.

Next to the steel building there is a reinforced concrete building. It is also frame structure (one bay, 12 m span). Because of the small gap between the two buildings as well as due to the common foundation used in row C the two buildings are analyzed together in order to account for the structure to structure interaction.

A comprehensive 3-dimensional FE model has been developed for all essential structural elements (fig.9). The soil-structure interaction is accounted by equivalent spring and dashpots, connected to each footing. The seismic excitation is defined by a maximum acceleration for 10^{-3} annual probability of exceedance and a site specific broad band design spectrum. Modal analysis and time history integration are used for the dynamic analyses. Static analyses for all design load combinations are performed also. Most unfavorable loading condition is found out and capacity estimation for all bearing elements is performed respectively. The results of the analyses are showing that:

1. All columns have sufficient bearing capacity and can withstand the design seismic motion.
2. The column-roof connection has insufficient capacity and bad detailing. Failure of the roof structure is a possible failure scenario.
3. The longitudinal beams and X-brancings are very slender and failure due to overloading is possible.
4. The safety of the foundation for sliding and overturning is guaranteed.

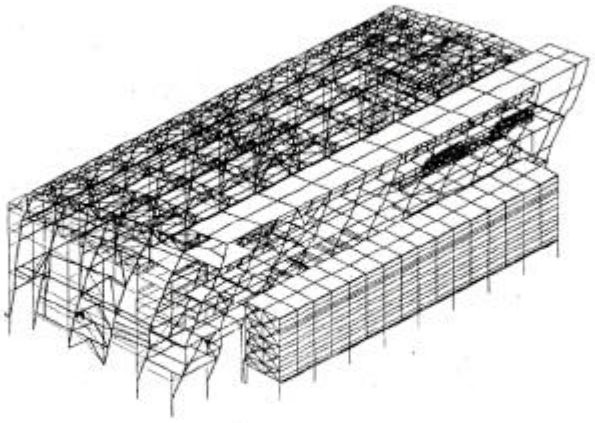

Fig. 9 Structure FE model, Second mode shape

Those conclusions mean that there are at least 4 scenarios with considerable influence on the overall structural safety that have to be analyzed. For each scenario a fragility curve is developed following the methodology described in section 3 above.

In order to improve the safety an upgrading concept has been proposed. It consist of improving of the column-roof connection by using a better detailing and reducing of the slenderness of the longitudinal beams and x-bracing by increasing the sections of the members or additional support points (shortening of buckling length). Fragility analyses are performed also for the upgraded structure and results are compared with the existing structure.

As explained in the beginning the fragility curve is assumed log-normal. The mean failure acceleration is determined by scaling the design acceleration by the factor of safety. In this analysis the following partial safety factors are used:

$$F=F_S.F_\beta.F_{RS}$$

Where

$F_{RS}=F_M*F_C$

F_M is safety factor representing the conservatism of the modeling.

F_C is safety factor representing the conservatism of the member force combination procedure.

Similarly the variations β_R and βu are determined for each partial safety factor.

As an example the values for the determination of the fragility curve for roof failure are presented below:

Safety Factor	Mean	β_R	βu
Fs	1.050	0.100	0.150
Fβ	1.400	0.050	0.100
F_M	1.100	0.050	0.050
F_C	1.100	0.050	0.050
F	1.791	0.132	0.193

The mean maximal acceleration that will cause failure is determined as A_H= 0.1*1.791=0.179g. The corresponding fragility curve is shown in fig. 10.

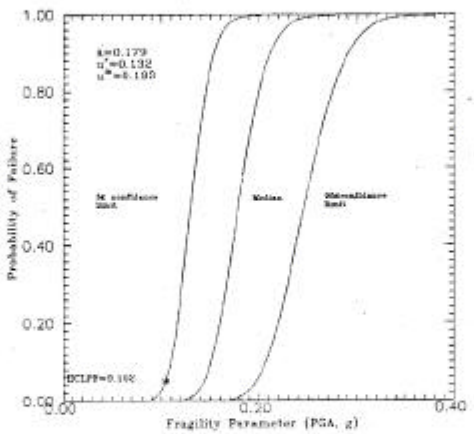

Fig. 10 Fragility curves, roof failure

Some of the considered failure modes before and after upgrading are presented in table 2.

Table 2. Fragility curve characteristics, turbine hall, VVER 1000.

	Before Upgrading			After Upgrading		
Failure Mode	A_m	β_R	βu	A_m	β_R	βu
Roof failure	0.179	0.132	0.193	0.356	0.132	0.193
Bracing failure– row A	0.125	0.132	0.193	0.404	0.132	0.226
Bracing failure– row B	0.147	0.132	0.193	0.390	0.132	0.226
Bracing failure– row C	0.154	0.132	0.193	0.396	0.132	0.226

For the site of that building there are seismic hazard curves for maximal acceleration developed. The methodology uses is the same as described in case 1. The fragility curves and the hazard curve are integrated together and the annual probability of failure is determined for each failure scenario. The results are presented in table 3 for 85% confidence limit.

Table 3. Probability of failure (85% confidence level)

	Before Upgrading	After Upgrading
Failure Mode	**Probability of failure**	**Probability of Failure**
Roof failure	1.242E-3	2.356E-5
Bracing failure– row A	7.872E-3	1.518E-5
Bracing failure– row B	3.548E-3	1.886E-5
Bracing failure– row C	2.836E-3	1.717E-5

4.3. Case 3. Probabilistic Liquefaction Analysis

There are big number of liquefaction analysis methods. Most of them are based on statistical observations. Unfortunately there are rear confidence limits and uncertainties reported for the respective method. In the presented case analysis a site was initially assessed deterministically using several methods. Some of the methods were assessing the site as potentially liquefable, some were saying that the site is safe. The characteristics of the site are presented in table 4. For example the simplified Seed method (1971) is saying that the site will liquefy. The Seed method based on SPT values (1979) is saying that the site is almost on the boundary, i.e. the safety factor is generally more than 1, but is not definitely safe. The Seed method (1979) based on SPT and experimentally determined shear stress ratios (3 axial tests) is saying that the site is safe. The pore pressure generation analysis (Seed 1976) is also saying that the site is safe.

Table 4 Site characteristics

Layer No	Material	Thickness	Weight density	Relative density	SPT (N)	Permeability
		m	t/m3	%	N	cm/s
1 *)	Sandy loes	8.6-10.5	1.9-2.1	-	-	
2	Loes-clay	1.7-3.5	1.9-2.1	-	-	
3	Fine sand	0.5-3.0	1.6-1.7	53	9/21	4*10-4
4	Fine sand	0.5-4.7	1.6-1.7	44	9/21	4*10-4

*) There is an overburden of 240 kPa on that layer.

As known the engineering measures to prevent liquefaction are very costly. In order to assess the severity of the risk and in order to prioritize the upgrading actions a probabilistic analysis has been performed. The analysis scheme is based on LHCED procedure, i.e. for the liquefaction assessment multiple analyses based on the Seed (1979) method are performed.

The safety factor for liquefaction is expressed according to Seed (1979) as:

$$FS=SR/L$$

The SR is the cyclic stress ratio for equivalent number of cycles Ne (soil resistance) and L is the stress ratio caused by the equivalent number of cycles by an earthquake (seismic load). The experimentally determined cyclic stress ratio to generate liquefaction for the site is presented in fig. 10.

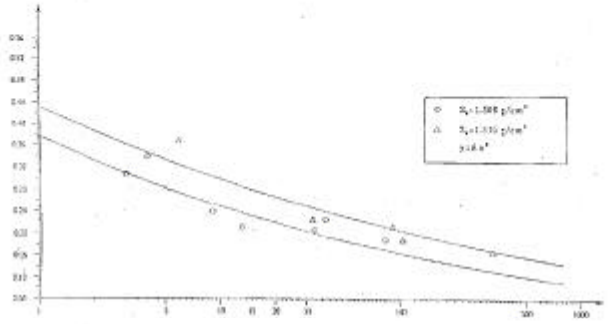

Fig. 10 Experimentally determined cyclic stress ratio

Because of the importance of the equivalent number of cycles of the earthquake excitations an additional simplified statistical analyses was performed to assess the site-specific excitation cycles. The site is located in the northern part of Bulgaria. The seismic hazard is governed by two type of seismic sources: local shallow sources with maximum magnitude up to 5.5 and fare field intermediate depth Vrancea source with maximum magnitude up to 7.5. The shallow source excitations were represented by 43 three component accelerograms recorded in Bulgaria, Italy, Turkey and Japan. The Vrancea excitations are represented by 9 three component accelerograms recorded in Bulgaria and Rumania during the 1977, 1986 and 1990 Vrancea earthquakes. The equivalent number of cycles is determined according to he Seed's procedure (1975). The results are summarized below:

Local sources	**Fare field sources**
Equivalent number of cycles	Equivalent number of cycles
Mean value: 5.778	Mean value: 10.34
Standard deviation 3.006	Standard deviation 6.08
Minimum value: 1.57	Minimum value: 2.82
Maximum value: 16.63	Maximum value: 30.62

The multiple deterministic analysis is based on the LHCED procedure. There are 10 samples generated and analyzed for 3 seismic levels (annual probability of exceedance 10-3, 10-4 and 10-5). The generation is done on the key parameters as: layer thickness, weight density, relative density, permeability, effective number of cycles, cyclic stress ration, etc.

For each set of variables the Seed's method for liquefaction analysis is applied deterministically for three levels of seismic excitation. The seismic levels correspond to annual probability of exceedance 10-3, 10-4 and 10-5. The seismic hazard curve for maximum acceleration is the one presented in Fig.1. In order to achieve better accuracy the layer 3 and 4 (where liquefaction is expected) are further subdivided in thinner layers (usually 0.5m thick). For each layer the central factor of safety and the corresponding standard deviation is computed for each seismic level. The conditional probability of failure is computed assuming that the resistance and loading are log-normally distributed. The computed conditional probabilities are presented in table 5.

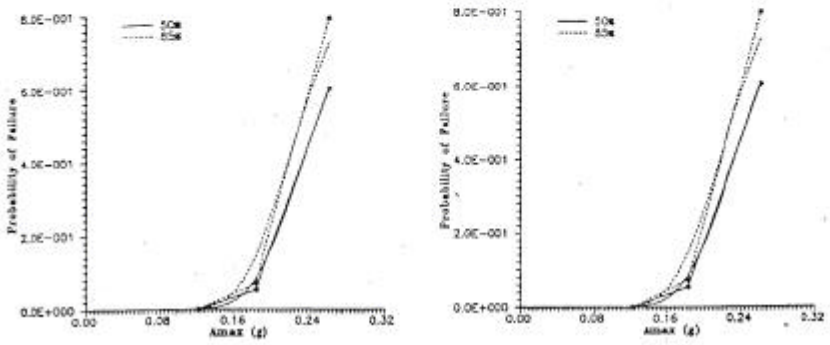

Fig. 11. Fragility curve generated by interpolation and extrapolation of the results for 4 layers

Table 5. Conditional probability of failure due to liquefaction of layers (sub-layers) 3 and 4

Layer No	Level 1 (10^{-3})	Level 2 (10^{-4})	Level 3 (10^{-5})
3.1	1.78E-10	5.07E-02	5.96E-01
3.2	2.24E-10	5.33E-02	6.05E-01
3.3	2.06E-10	5.51E-02	6.11E-01
3.4	2.34E-10	5.71E-02	6.17E-01
4.1	2.62E-10	5.81E-02	6.21E-01
4.2	2.59E-10	5.69E-02	6.23E-01

Based on those conditional probabilities a fragility curve is generated by interpolation and extrapolation of the results for each layer (sub-layer), Fig.11. The fragility than is convoluted by the hazard curve and the probability of failure due to liquefaction for each layer (sub-layer) is determined (table 6).

Table 6. Probability of failure due to liquefaction of layers 3 and 4

Layer No	Probability
3.1	6.71E-05
3.2	6.89E-05
3.3	6.98E-05
3.4	7.07E-05
4.1	7.18E-05
4.2	7.19E-05

5. Conclusions

The tools usually applied for probabilistic safety analyses of critical structures could be used very efficiently in the seismic re-evaluation. The key problems are the seismic hazard definitions and the fragility analyses. The fragility could be derived either based on scaling procedures or on the base of generation. Both approaches have been presented in the paper. Unfortunately the VVER plants usually do not have sufficient design documentation and scaling could not easily be based on original design data. Both for scaling and generating purposes careful modeling and analyses are needed.

After the seismic risk (in terms of failure probability) is assessed there are several approaches for risk reduction. Generally the methods could be classified in two groups. The first group comprises the methods for monitoring and control. Generally their aim is to collect additional information and based on that to

improve the assessment. The second group of measures is the engineering ones. The engineering includes the repair, strengthening and upgrading of the investigated systems. In all cases the risk assessment is a power tool for decision taking.

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SEISMIC SAFETY EVALUATION AND ENHANCEMENT AT THE PAKS NUCLEAR POWER PLANT

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ABSTRACT

A comprehensive program for the evaluation and enhancement of seismic safety of the Paks Nuclear Power Plant was started in 1993 and now is in progress. In the first two years of the program the urgent and quickly realizable fixes were performed, seismic instrumentation was installed, and a procedure was elaborated for the activities to be performed after a possible earthquake. Re-evaluation of the site seismic hazard was finished in 1996. The adequacy of the evaluation was approved by micro-seismic monitoring results in 1998. A concept has been established for the safe shutdown and cooling-down of the reactor core, which is based on reinforcement of systems and minimizing system modifications. The systems, equipment and structures relevant for seismic safety have been determined on the basis of this concept. Seismic re-qualification of the main part of the equipment and structures has been completed, now design and implementation of the reinforcements is in progress. Essential part of upgrades is already implemented. A level 1 seismic PSA has been started for the quantitative characterization of plant safety achieved after implementation of upgrading measures and for the confirmation of the safety concept and plant actions in case of earthquake. The programme has to be finished by the end of 2002. The paper gives an overview of the seismic safety evaluation and enhancement program for the Paks NPP. The specific aspects of the re-evaluation and upgrading are discussed in details.

1. INTRODUCTION

Seismic problem of the WWER type units is derived from the fact that during their design the seismic hazard of the sites was underestimated and safety aspects related to the external events were neglected [1]. In the case of Paks NPP the site seismic hazard was underestimated, so the units of WWER 440/V213 type of Paks NPP were not designed for any earthquake loads. At the end of 1980s it was clear that seismic hazard of the Paks site can be much greater, than it was assumed during the design. In 1993 the NPP launched a comprehensive program for seismic assessment and upgrading of the plant which is due to be implemented on all of the units by the year 2002 [2].

During last two decades essential experience has been obtained during seismic re-evaluation and upgrading of large number of operating NPP, including WWERs too. The re-evaluation and upgrading of Paks NPP is going in line with the international practice. Nevertheless the experience of Paks NPP is unique form the point of view that, the Paks NPP has no seismic design and qualification at all. It means the basic goal of the Paks NPP seismic safety programme is to establish the seismic design basis. Consequently, there are aspects of the seismic re-evaluation and upgrading of Paks NPP, which required specific considerations and solutions, e. g.:

- evaluation and upgrading philosophy and basic requirements for a plant not designed for earthquake,
- applicability of the re-evaluation and design methods developed in western countries for a nuclear power plant designed and built according to Soviet standards,
- development of an rational approach for upgrading, feasibility aspects.

2. SCOPE OF THE SEISMIC SAFETY PROGRAM

The essential safety requirement for the nuclear power plants is to ensure the shutdown and stable sub-criticality of the reactor, its cooling-down and decay heat removal function, and limitation of radioactive releases.

The seismic safety program includes the following tasks:

- evaluation of seismic hazard of the site, which includes the geotechnical survey of the site, analysis of soil liquefaction, etc.
- establishing the safe shutdown and heat removal technology, and elaboration of the list of the structures, systems and equipment, which are essential to ensure seismic safety,
- installation of seismic instrumentation and elaboration of pre-earthquake preparedness and post-earthquake actions,
- evaluation of the seismic capacity of systems and structures relevant for safety,
- performing the necessary upgrading measures taking into account the priorities and the feasibility of the necessary corrective actions.

Seismic re-evaluation and upgrading of the NPP have to be carried out for the design base earthquake (DBE).

The implementation of the seismic safety program is harmonized and synchronized to the implementation of other safety upgrading measures, which may also affect the seismic safety of the plant. For example, the ongoing reconstruction of the reactor protection system or the relocation of the emergency feedwater system from the longitudinal gallery building to a safe position under the localization towers.

3. SITE SEISMIC HAZARD RE-EVALUATION

The re-evaluation of site seismic hazard was completed in 1996. The design base earthquake with 10^{-4} annual recurrence frequency has been determined by probabilistic methods. The best estimate, uniform hazard response spectrum (UHRS) is obtained for the design base earthquake level. The free field response spectra were obtained by a non-linear calculation, considering the loose ground layer of the top 30 meters. The horizontal PGA of the design base earthquake is 0.25 g, which is ten times greater than it was assumed earlier during the design. The comparison of the PSHSA PGA value with the 84% confidence level PGA defined in a deterministic way shows good agreement.

The soil on Paks site is loose, the shear wave velocity in the top 30 meters sandy layer is about 300 m/s, and the ground-water table is high. Consequently, the possibility of the soil liquefaction had to be evaluated. It was found, that only the layer under the free surface between the depth of 10 and 20 meters is disposed to be liquefied. The return period of liquefaction calculated by a probabilistic method is between 11000 and 14000 years. The probability of the liquefaction under the nuclear power plant main building is significantly less, its recurrence frequency is estimated to be between 15000 and 18000 years. It means that in case of a design base earthquake no global liquefaction can be expected.

Prior to obtaining the above mentioned results a conservatively determined reference level earthquake has been defined for the preliminary seismic margin evaluation and design of urgent and easy-to-perform fixes. For qualification purposes the NUREG/CR-0098 soft site, median response spectrum was selected for the

PGA of 0.3 g. Meanwhile for design of the fixes the US NRC Regulatory Guide 1.60 response spectrum was used for the value of 0.35 g.

4. SAFE SHUTDOWN AND HEAT REMOVAL CONSIDERATIONS

The basic safety requirement for nuclear power plants to ensure shutdown and cooling down of the reactor, decay heat removal and to restrict radioactive releases after an earthquake. In case of Paks NPP the long-term heat removal is required which is not limited to the 72 hours like in typical SMA/IPEEE programmes. The control of the critical plant parameters as well as the radiation conditions has to be ensured, too. Furthermore it is required to ensure the level of redundancy (3x100%) corresponding to the design philosophy of the plant with conformance with single failure criteria.

Procedure for safe shutdown, cool down and long term heat removal for NPP Paks (called seismic safety technological concept and referred to as SSTC hereinafter) has two purposes:

- to identify structures, systems and equipment necessary for safe shutdown, cool-down and long term heat removal, as well as for monitoring the plant status;
- to determine operator actions and outline operational as well as administrative procedures and provisions required to achieve and maintain safe shutdown conditions following a design basis earthquake.

The seismic safety technological concept has been elaborated in two versions so far.
The first version was established when, on the basis of the preliminary hazard assumption, a PGA value of 0.35 g had to be considered. Reinforcements and fixes for this high load level are considerably expensive and seem to be very difficult to implement. It was preferable to choose those systems for safe shutdown and cool down which are inside the reinforced concrete part of the main building, because only this part of the building has adequate load-bearing capacity. Fast closing valves have to be used for the separation and isolation of the systems, which are not reinforced, from the reinforced ones. Cooling down of the reactor could be performed by the use of secondary bleed and feed. Long-term heat removal would have been performed by the heat exchangers of the low-pressure emergency cooling system (LPECCS), which should be modified too for ensuring this function. In this concept safety system modifications and installation of a large number of isolation valves would cause serious difficulties. Most critical structure in this case is the longitudinal gallery building housing many systems and equipment vital for safety.
While evaluating the main building structure and equipment and designing the system modifications for the final seismic input of 0.25g, it have been recognised that

- the modification of the safety systems and installation of isolation valves is not only expensive, but it can reduce safety in all other cases than an earthquake,
- the best technical solution for fixing the longitudinal gallery building could be done best by reinforcing the steel frames of the turbine hall and the reactor hall.

The last was the most conclusive recognition, which led to reconsider the concept of the safe shutdown and heat removal procedure. The new concept is using necessary systems and equipment in the turbine hall, since it will be fixed and the systems placed in the turbine hall could be considered as available for the heat removal after an earthquake. Of course these systems have to be fixed too.

Since the critical path in the realization of upgrading measures will be the implementation of structural fixes, the additional equipment and piping fixes in the turbine hall can be performed without delaying the program.

The new concept of safe shutdown and cool down elaborated in 1998 is based on the fact, that reactor shutdown, cooling down and its long term cooling can be realized by the systems and in the way, that it is performed in other cases than an earthquake. The equipment and piping fixes are less expensive than the separation by valves. Consequently, the systems should be reinforced until the natural system boundaries, so that only minimal number of isolation valves has to be installed. In some cases the ALARA principle has to be taken into account while selecting the separation or the fixing of a system or part of it. According to the new concept there is no need for automatic actuation of specific functions, like isolating valves after an earthquake.

This re-qualification and implementation of the necessary reinforcements compose the seismic safety program. While evaluating the load-bearing capacity of the systems and structures the seismic interactions have to be considered.

The concept of reactor shutdown, cooling down and long term cooling will be qualified best by a seismic PSA. This work is finishing in 2001. Specific aspects of the WWER-440/V213 seismic PSA and also the preparatory work will be discussed below.

5. SEISMIC CAPACITY RE-EVALUATION AND ELABORATION OF REINFORCEMENTS

5.1 Approach and methodology

As the main objective of the seismic safety programme is to establish the design base for the case earthquakes at an operating Soviet designed NPP, the approach of evaluation and upgrading and the selection of rational methods have large importance. Formally the design base requirements should be fulfilled by straightforward design procedures, which may lead to conservative results, heavy upgrades and feasibility problems. The great advantage of the seismic margin assessment methodology (SMA) and the experience based evaluation and qualification methods (GIP, GIP-WWER) were recognised. Therefore a graded approach has been elaborated and adopted. According to this the design procedures, standards and criteria have to be applied to the ASME Class 1 and 2 equipment and piping and also the confinement part of the main reactor building. Otherwise less conservative approach might be applied based on realistic damping values and accounting ductility when appropriate. The empirical methods are used for the equipment (functionality) qualification and simplified qualification procedure was elaborated for the low-energy and small diameter pipes. The applicability of these re-evaluation methods was carefully studied on the basis of systematic evaluation and comparison of US, German and Soviet design requirements and procedures. Although the liberal approaches may be followed while evaluation the capacity of structures, systems and equipment, the design of upgrading measures has to be performed according to procedures ensuring code compliance.

5.2 Evaluation and Upgrading of Structures

Design of WWER-440/V213 type twin units has several features, which determine the as built seismic resistance of the units. The box-like reactor building made of reinforced concrete (the containment) was designed for an overpressure of 0.15 MPa, so this building bears the loads caused by a design earthquake. The steel frame turbine hall of 39 m span is connected to the longitudinal gallery building, which is attached to the rigid reinforced concrete part of the confinement. The beams supporting the floors of the

gallery building are connected to the wall of the reactor building and the pillars of the turbine hall. The reactor hall, turbine hall and gallery buildings are covered with concrete roof slabs. Seismic resistance of the brick walls separating the different rooms of the gallery buildings is inadequate.

The main building is a set of coupled structures having a separate foundation and widely varying rigidity, and the distribution of the stiffness and masses is highly complex. The problem of optimal modeling of coupled structures with very different characteristics and also the adequate modeling of twin main buildings on a common base mat had to be solved [3], [4]. In the case of the main building structure the soil–structure interaction is modeled through the introduction of the frequency dependent dynamic stiffness matrix obtained for all points of the structural model in contact with the soil, and the equations of motion are solved in the frequency domain. This approach leads to an essential reduction in conservatism compared with the routine calculation methods [5].

It seemed to be the most beneficial to stabilize the longitudinal gallery by reinforcement of steel framework of the reactor hall and turbine hall. The idea is to transfer the transversal load from the turbine hall, intermediate building (transverse gallery) and reactor hall to the very rigid reinforced concrete localization towers. This means reinforcement of the roof bend in order to acquire a disk-behavior, reinforcement of the cross braces of the columnes, and transfer of the transversal forces to the localization tower and to a bridge construction connecting them. Thanks to this in the gallery building, which is overfilled by equipment, there is no need to implement modifications and reinforcements. This reinforcement concept allowed considering also the systems in the turbine hall for cooling and long term heat removal as it was mentioned above. The structural fixes of the turbine and reactor halls excluded also the falling-in of the concrete roof panels. Solutions for increasing of seismic resistance of the structures essentially mean use of new structural elements (e.g. cross braces, reinforcement of the joints) and reinforcement of the main load bearing elements. The implementation of the structural fixes is going on. The total weight of the reinforcement to be added is more than 1700 t.

Qualification of the auxiliary buildings, diesel engine rooms and water intake, and development of the reinforcement have been also carried out.

Although global soil liquefaction should not be expected, in case of connected buildings with separated foundations, such as the main building-longitudinal gallery-turbine hall, it is reasonable to examine the possibility of local liquefaction and relative settlement of the buildings. Such examination of the main building complex has to be performed, too.

Evaluation of the exhaust stacks should be additionally mentioned, since they need another reconstruction due to the construction deficiencies of that time. It has been found that the load-bearing capacity of the structure restored by the reconstruction will ensure that the exhaust stacks do not throw on the diesel engine building. This is also an example how the various safety upgrading or reconstruction measures interact.

In 1993-1994 the non load-bearing structures, mainly brick walls of the gallery buildings have been reinforced in the frame of the easy-fix project in order to protect the safety systems located there from falling-on.

5.3 Qualification and upgrading of equipment, piping and I&C

According to the graded approach of the seismic evaluation dynamic analysis and code compliance have been required for the most important piping and equipment. For dynamic analyses of the primary circuit (loops, steam generators etc.) an integrated model has been used, which includes the reinforced concrete structure of the reactor building together with the primary loops and equipment [6], [7]. Its purpose was to

qualify the primary system for a less conservative seismic excitation, and to receive relative displacements of the primary circuit in order to evaluate the possible impacts. The capacity evaluation of the primary system equipment has been performed in compliance with KTA standards. Other piping and equipment in the confinement have been evaluated using also KTA but with certain realistic assumption on the damping and ductility. For the piping and equipment outside of confinement the SMA type evaluation technique has been applied. The small bore low energy and cold large bore pipes have been evaluated using simplified calculation and walkdown based method. The upgrades design always follows code requirements. The experience of the re-evaluations shows that the insufficient seismic capacity of essential components is due to their anchorage, since during their design the earthquake loads were not considered. The analyses indicate that the distances at the pipeline supports are too large, so additional supports and viscous dampers have to be installed. The upgrading of the primary system by viscous dampers was developed and implemented. Fixing of other piping systems have been performed by additional supports and also viscous dampers.

For evaluation of functionality of active equipment (pumps, motors, valves, breakers, etc.) empirical methods were used. The qualification is mostly completed with considerably favorable results.

A main conclusion of the easy-fix project was, that anchorage of the I&C racks and cabinets, accumulators is not adequate. Practically all the safety related electrical and I&C cabinets had to be reinforced with new anchorage at the bottom or with cross brace at the top. Distances of the cable tray supports were too large in all cases and additional supports had to be installed. Due to the weak anchorage, additional anchorage had to be used at some mechanical equipment. The easy-fix project was completed by 1995. The easy-fixes including reinforcement of the brick walls concerned more than 5500 elements on the four units and were accompanied by building in steel framework of about 450 t.
The easy-fix reinforcement mainly ensures structural safety and stability of the racks and cabinets, and protects them from falling down of a brick wall. In most cases reinforcement is sufficient already for the functionality of the equipment, but it has to be checked. In the frame of a PHARE project completed in 1998 the qualification of the functionality of electrical and I&C equipment has been performed using empirical methods [8]. Additional fixes are needed in some cases in order to ensure functionality. There are some items, which cannot be qualified empirically. For these ones qualification specification was prepared in the frame of the above PHARE project [8]. Specifications for altogether 665 elements have been elaborated, among them there are 12 rack specifications, 647 relay and 4 tank qualification specifications. The qualification is going on in accordance to the specifications. Main achievement of the PHARE project was the systematic examination relating to behavior of relays during an earthquake and their consequences. Performing the above mentioned electrical and I&C qualifications and implementing the measures resulted from them the seismic re-qualification of safety systems will be finished. Here it should be mentioned that during the design and qualification of the new reactor protection system the actual seismic input was taken into account.

6. PRE-EARTHQUAKE PREPAREDNESS AND POST-EARTHQUAKE ACTIVITIES

In 1993 seismic instrumentation was installed at each unit of the NPP. The system registers the response signals generated in typical points of the reactor building and the free field acceleration signal in two places at the site. The question of automatic reactor shutdown by the personnel was decided between 1993-1997. Recently the procedure based on the response spectrum and cumulative absolute velocity criteria has been introduced. The emergency operating procedure of the Paks plant contains the activities of the personnel during and after an earthquake. An overall guideline has been prepared for determining plant status after an earthquake.

7. SEISMIC PSA

A level 1 seismic PSA is in preparation and is an integral part of the seismic safety program of the Paks NPP. Currently level 1 PSA models and results are available for internal initiating events, internal fires and flooding at full power [9-11]. Internal initiating events for off-power operating modes of the plant have also been analyzed by level 1 PSA [12]. Quantification of seismic risk will be a substantial extension to the existing level 1 PSAs for Paks, and the majority of important initiating events will have been taken into account by the end of the study because earthquakes are considered to be the only risk significant external hazard.

The objectives of the seismic PSA have been defined in accordance with the international practice and the level 1 PSA of Paks that is available for other types of initiating events. These objectives are as follows:

- Quantification of frequency of core damage from seismic events;
- Identification of important contributors to seismic risk (initiating events, accident sequences, component failures, human errors);
- Identification and ranking of plant vulnerabilities based on qualitative insights and quantitative results of the study,
- Confirmation of the results of the seismic safety programme;
- Development of recommendations for safety improvement including feedback to the seismic safety program.

The major steps of the Paks seismic PSA have been decided in accordance with most common international practices [13] and methodologies applied recently in most OECD countries [14], including extension of the site seismic hazard study up-to low probability events and local effects.

A note should be made on treatment of containment failures (i.e. failures of the so-called hermetic rooms) within the seismic PSA for Paks. It is an important subtask which is to provide an interface to future extension of the level 1 seismic PSA to level 2 analysis by identifying and modeling those seismic failures of structures and equipment that lead to a containment bypass sequence. The analysis covers failures of containment isolation and the cooling system without considering containment loads due to thermal-hydraulic effects and accident progression. For this purpose the existing system models should be extended with failures of containment isolation valves, the containment spray system and the associated electric power supply and I&C subsystems. Structural failures affecting the integrity of the reactor building confinement box and the bubble tower system will be determined. Failures of containment structures and systems will be incorporated into the event sequence models as a separate heading by using the results of fragility evaluation to describe the probabilities of seismic failures in a way consistent with the rest of the seismic PSA model. In accordance with the PSA for internal initiators, data on random equipment failures of containment systems will be based on a combination of generic and plant specific reliability data.

Core damage frequency due to seismic events will be calculated by the use of a standard algorithm for processing event trees and fault trees in a PSA, i.e. the minimal cut sets of earthquake induced accident sequences and their frequencies will be determined. Seismic risk will be estimated for each acceleration range and an overall value of core damage frequency will be determined through integrating the results obtained for the various acceleration ranges. Frequency of containment bypass sequences will also be quantified.

Uncertainty analysis will be carried out to calculate density and cumulative distribution functions of core damage frequency using Monte Carlo sampling of uncertainty distributions for seismic and random

failures. Measures of importance and sensitivity will be produced for important initiating events, equipment failures and human actions/errors. Qualitative analysis of important seismic accident sequences and quantified risk measures are expected to be used jointly in support of identifying plant vulnerabilities and developing recommendations for further enhancing of the seismic safety, if needed.

8. CONCLUSIONS

Main achievements in the evaluation and improvement of seismic safety of Paks NPP can be summarized in the following.

The reinforced concrete part of the reactor building, which is the containment of the reactor, has adequate seismic capacity. The gallery buildings connected to the reinforced concrete part of the reactor building, the reactor and turbine halls need essential upgrades. The technical solutions to increase seismic resistance of these structures have been elaborated and their implementation is under process. Nearly all of the non-structural walls have been fixed. Qualification of the building other than the main building is also completed, now the design phase of the reinforcements and preparation of their implementation is under process.

The equipment and pipelines in the primary system have significant as built seismic capacity, but in order to limit stresses and displacements viscous dampers were installed in 1998. Qualification of the technological system components is completed, now the design phase of the reinforcements and preparation of their implementation is in progress.

Anchorage of the racks of the damaged electrical and I&C equipment was already completed in the frame of the easy-fix project. In 1998 qualification of functionality of the elements was completed by an empirical method. The qualification specification for the outliers has been developed and qualification is being carried out.

The NPP has adequate seismic instrumentation, and a detailed procedure exists for human activities after an earthquake.

It is expected that the seismic PSA will verify the concept of reactor shutdown, cooling down and constant cooling, and effectiveness of the complex reinforcement program. By performing seismic PSA for the Paks NPP level 1 PSA results will be representative for most initiating events of safety concern including internal initiators and external hazards.

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SEISMIC RE-EVALUATION PROGRAM OF THE ARMENIA NUCLEAR POWER PLANT
– Results from an international co-operation project –

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ABSTRACT

The Armenian Nuclear Power Plant, constituted by two WWER-440 type reactor, was shutdown in 1989 after the destructive Spitak earthquake (December 1988). Following the decision to re-start the operation of Unit 2 (April 1993), Armenian authorities launched a complete program for enhance the plant safety.

A co-operation effort, financed by a number of different Organisations, among which EC through TACIS Program, IAEA and U.S. DOE, aiming at improving the seismic safety of the plant was undertaken in accordance to methods and criteria as generally accepted by the international community, taking into account that no regulatory guides exist on seismic re-evaluation of existing NPP. An important milestone of this co-operation effort was the conference hosted by IAEA in Vienna in 1999 among the donor Institutions and involved Organisations where a joint workplan for the plant seismic re-evaluation, to be completed soon on the base of the results achieved, was agreed. On this basis different Organisations, with very different approaches to the assistance to developing countries (e.g.: bilateral, UN, EC, etc.), co-operated within the same framework in order to provide the best safety improvement to the plant with a global resource optimisation. As a result, the planned tasks are under implementation and safety review, with a very important contribution by the Armenian Nuclear Power Plant (ANPP) personnel which has been trained and involved since the very beginning of the project. The agreed technical approach among the different Organisations was codified in a IAEA Technical Guidelines Document, endorsed by both the ANPP and the Armenian Safety Authority (ANRA).

In the above mentioned framework three main aspects were addressed:

- re-assessment of the site geological stability and seismic hazard aimed to define the site specific maximum earthquake (*Review Level Earthquake*) to be used for plant re-evaluation purposes
- assessment of the seismic capacity of as-built structures, systems and components (SSC) essential to achieve the plant safe shutdown
- plant upgrading

The first aspect was developed in compliance with criteria established for new nuclear power plants (IAEA Safety Guides 50-SG-S1). Earthquake peak ground accelerations of 0.21g and 0.35g were obtained for two different fractile values, respectively 50% and 84%. For re-evaluation purposes a broader band ground response spectrum than the site specific one with a free field horizontal PGA of 0.35g was chosen.

A detail work plan for the seismic capacity assessment of SSC has been set-up. The seismic response evaluation of buildings and structures of Unit 2 is firstly performed to determine seismic forces and the in-structure input motion to systems and components. Seismic resistance of distribution systems and components is then assessed basically relying on the seismic margin assessment method. The scope of this safety re-evaluation process is determined by the Safe Shutdown Equipment List (SSEL) and plant walkdowns.

INTRODUCTION

BACKGROUND

The Armenian NPP of Medzamor is situated in the southwest part of the republic of Armenia, 28 km from the capital Yerevan, at about 950 m above mean sea level. The plant is constituted by two 408 MWe (376 MWe net) reactors based on the VVER 440 - V230 Soviet type model. The design of the standard V230 was modified due to site conditions and local seismicity, leading to the construction of an original model called V270. The construction started in 1969 and Units 1 and 2 were in operation in 1976 and 1980 respectively. Unit 2 at the time of the Vrancea Earthquake (March 1977) was in the early stage of construction and it was redesigned to be "earthquake-proof".

The plant continued to be in operation without any visible damage after the strong earthquake in Spitak in December 1988. However, owing to safety concerns related to the high seismicity of the plant's location, Unit 1 and 2 were respectively closed in February and March 1989.

In April 1993 the Armenian government decided to start the preparatory works for the reopening of the Unit 2, being the newer of the two, due to the energy shortages in the Republic of Armenia in the early nineties. An international co-operation effort was then initiated to assist Armenia in improving the safety condition of the Medzamor NPP with the purpose of mitigating the critical energy situation of the country. In November 1995 that Unit came back on line, providing about 35% of the nation's electricity.

The first seismic safety mission was conducted in May/June 1992 in the framework of an overall study on energy situation carried out by World Bank at the request of the G-7 (Group of Seven) and with the IAEA. As a conclusion it was recommended to the Armenian authorities that three major aspects had to be dealt with if the decision to restart the plant operation would be taken:

- verification of the geological stability of the site,
- determination of the site ground motion for the re-evaluation of the plant's structures, systems and components,
- development of a complete program for the re-evaluation of the seismic capacity of structures, systems and components important to safety.

In 1993 Armenia joined the International Atomic Energy Agency and a technical assistance and co-operation project on seismic safety was agreed in September 1993. Since then, many Seismic Safety Review Missions have been carried out in order to review the situation and progresses in the field of the above mentioned issues and plant operators were trained on all the aspects of the seismic safety.

In February 1997 an IAEA Technical Guidelines document [2] was finalised on a site/plant specific basis and it was approved in March 1999 by the Armenian Regulatory Body (ANRA) and plant operator (ANPP) as reference document in the seismic re-evaluation of the Armenian NPP. As neither the IAEA or any regulatory authorities has established official and comprehensive codes for the seismic re-evaluation of existing NPP, the above document has the purpose of providing the general framework within which the seismic re-evaluation program has to be carried out in a manner consistent with methods and criteria as recognised in the international practice.

In early 1994, Armenia and Russia agreed to co-operate in restarting the plant: the proposed activities included a full site investigation, improvements in safety standards and maintenance and repair. In January

1995, Russia's Duma ratified an agreement for a credit line to Armenia for the restart of the Medzamor NPP to be earmarked for Russian technical expertise, fuel and equipment.

In 1996, following a study on the major safety issues of Medzamor NPP performed by the European Bank for Reconstruction and Development (EBRD), Armenia was retained eligible for grants for safety assistance. The European Union allocated a budget under its TACIS program for safety related up-grades at Unit 2, most of which have been spending for on-site assistance provided by a consortium of European utilities led by Italian SOGIN (former ENEL/SGN).

In 1996 United States began working on co-operative safety projects with Armenia in the framework of a comprehensive assistance policy to reduce risks at older Soviet designed NPP.

In 1994 Armenia established a nuclear regulatory body – the Armenian Nuclear Regulatory Authority (ANRA). Since then ANRA has been provided with training in all safety related activities, including seismic issues, in the frame of U.S. and IAEA assistance programmes. In addition, EU is supporting the Armenian Safety Authority by means of a consortium of Western European technical safety organisations, led by Riskaudit, that is providing assistance to ANRA in licensing procedures for up-grades proposed by ANPP and its consultants.

An important milestone of the co-operation effort outlined above was the co-ordination meeting hosted by IAEA in Vienna in 1999 among the donor Institutions and involved Organisations where a joint workplan for the plant seismic re-evaluation, to be completed on the base of the results achieved, was agreed. On this basis different Organisations, with very different approaches to the assistance to developing countries (e.g.: bilateral, UN, EC, etc.), co-operated within the same framework in order to provide the best safety improvement to the plant with a global resource optimisation. As a result, the planned tasks are under implementation and safety review, with a very important contribution by the Armenian Nuclear Power Plant (ANPP) personnel which has been trained and involved since the very beginning of the project.

THE ORIGINAL SEISMIC DESIGN

The seismic design basis was originally related to a seismic intensity of 8 on the MSK scale. Due to the lack of specific codes on seismic safety for nuclear facilities, the plant was designed according to the existing rules for conventional building. During the design stage and construction seismic loads were increased from 2 to 4 times using a maximum ground acceleration of 0.4g for the reactor shaft and 0.2g for the reactor compartment and an amplification factor up to 3 was used for analytical and experimental qualifications of primary system and other safety related equipment [1]. Supplementary modifications were made after the Vrancea earthquake in Romania (March, 4th 1977) [2].

In 1987, with the issue of the Russian design code for seismic safety of nuclear installations (PNAEG-5-006-87) a programme for seismic re-evaluation and up-grading was initiated. A first set of FRS for the Armenian NPP was calculated by the Atomteploenergoproject (ATEGP) – Nizhny Novgorod (Report A-34988) for equipment and distribution system seismic qualification. It was used four records form California earthquakes and one artificial accelerogram, all scaled to 0.2g in the horizontal direction and 0.1g in the vertical one. The spectra were generated using a simple beam model including the reactor compartment. The elasticity of the foundation was taken into account by spring elements.

The re-evaluation program continued also after the Spitak earthquake and during the shutdown period, even though with limited activities, up to the date of the plant reopening. The seismic intensity at the plant location during the Spitak earthquake was about 6-7 on the MSK scale and the peak ground acceleration recorded was approximately 0.06 g.

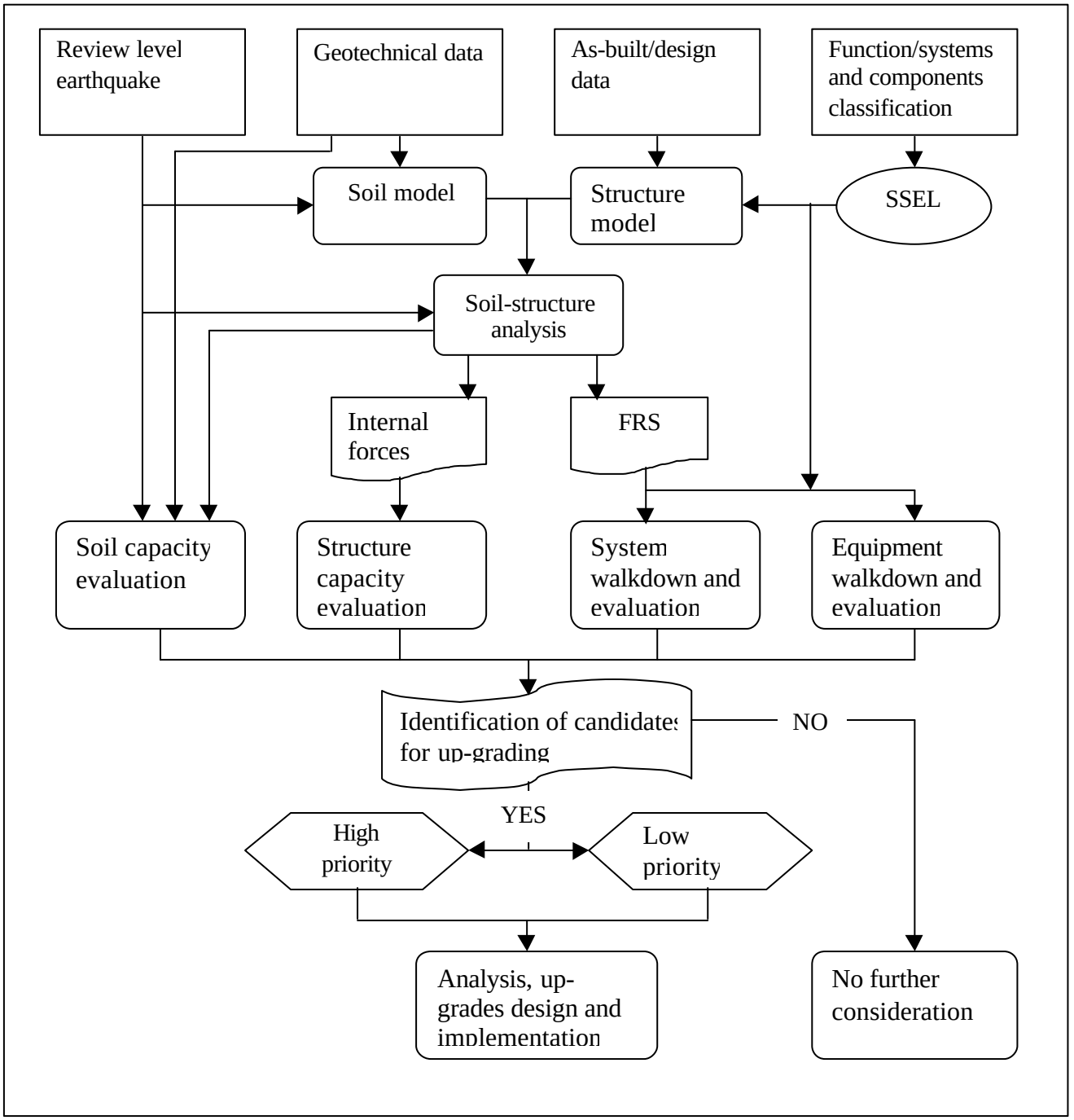
THE SEISMIC RE-EVALUATION PROGRAM

In the framework of the international co-operation effort mentioned above, the Armenian government decided to launch a complete programme for seismic re-evaluation of Medzamor NPP aiming at assessing and improving the seismic safety of the plant in accordance to methods and criteria as generally accepted by the international community. The program, implemented in a phased approach, addresses the following three main topics:

- re-assessment of the site geological stability and seismic hazard aimed to define the site specific maximum earthquake (*Review Level Earthquake*) to be used for plant re-evaluation purposes
- assessment of the seismic capacity of as-built structures, systems and components (SSC) essential to achieve the plant safe shutdown
- plant upgrading

A workplan for the seismic re-evaluation program, grounded on technical bases, was defined in the IAEA Technical Guideline [1], highlighting phases and tasks to be performed as well as sequences and interdependence among them.

Figure 1 – seismic re-evaluation flowchart



A detailed planning of each phase of the program is being developed by ANPP, taking into account also relationship and impacts on other projects relating to NPP safety improvements, such as the LBB project.

ANPP have collected the geo-technical and structural data, including implemented up-grades, and is presently organising them in a consistent and comprehensive manner for the ongoing and planned activities. At this purpose a spatial geotechnical model of the site is being developed.

RE-ASSESSMENT OF THE SEISMIC HAZARD

The main aim of the re-assessment of the seismic hazard was the determination of the site specific

maximum earthquake having a very low probability of being exceeded during the life time of the NPP; it is defined as *Review Level Earthquake*. As reference standard and code for seismic hazard evaluation the IAEA-SG-S1 (rev.1) was adopted [3].

A wide site investigation dealing with geology, geophysics, seismology, geotechnics, volcanology was carried out in years 1993 – 1995 by Armenian institutions and foreign organisations with an acceptable level of quality and completeness.

Earthquake peak ground accelerations of 0.21g and 0.35g were obtained for two different percentile values, respectively 50% and 84%. Specific ground response spectra, showing a rather narrow spectrum and having a maximum amplification of 2.3 centred at 5 Hz, were obtained. For re-evaluation purposes, considering the site characteristics, a broader band ground response spectrum based on the 50th percentile response spectrum for rock site given in the USA-NUREG/CR-0098 [4] with a free-field horizontal peak ground acceleration of 0.35g was chosen. The RLE as above defined corresponds to the SL-2 given in the IAEA SG-S1 (rev.1). The vertical acceleration was taken equal to 2/3 of the horizontal one in the entire frequency range.

As far as the background seismicity it was considered a local earthquake having a magnitude of 5.5 at a distance of 2.5 km and a depth of 10 km. The local earthquake spectrum is bounded on all the frequency range by the RLE; therefore it was not considered in the re-evaluation analysis of Unit 2.

EVALUATION OF THE SEISMIC CAPACITY OF THE UNIT 2

The evaluation of the seismic response of the NPP Unit 2 and consequently the capacity of buildings and structures to withstand the induced loads is presently underway. Analyses have been carried out by NIAEP (Russian Institute - Atomenergoproject Nizhny – Novgorod) with the supervision of the ANPP experts and are currently under review by IAEA.

In the co-ordination meeting held in Vienna on 1999, in the context of the joint workplan for the plant seismic re-evaluation, it was agreed to assist NIAEP in performing the evaluation of the Floor Response Spectra (FRS) and seismic capacity of buildings and structures as well as to complete the collection of data for the geotechnical characterisation of the site. EU/TACIS is evaluating the possibility to carried out both the activities by means of a specific contract. Technical Specifications for such activities have however already been developed by SOGIN, the EU/TACIS on-site assistance utility.

The seismic analysis of the plant will be conducted on those buildings and structures, whose resistance is essential to achieve the safe shutdown of the NPP Unit 2, by using realistic as-built properties and data, having considered all the plant up-grades to the original design. These are the Unit 2 main building, including reactor building, longitudinal and transversal electric shelves building, ventilation centre, boron injection centre and turbine building as well as the diesel generator building (figure 2 [1]).

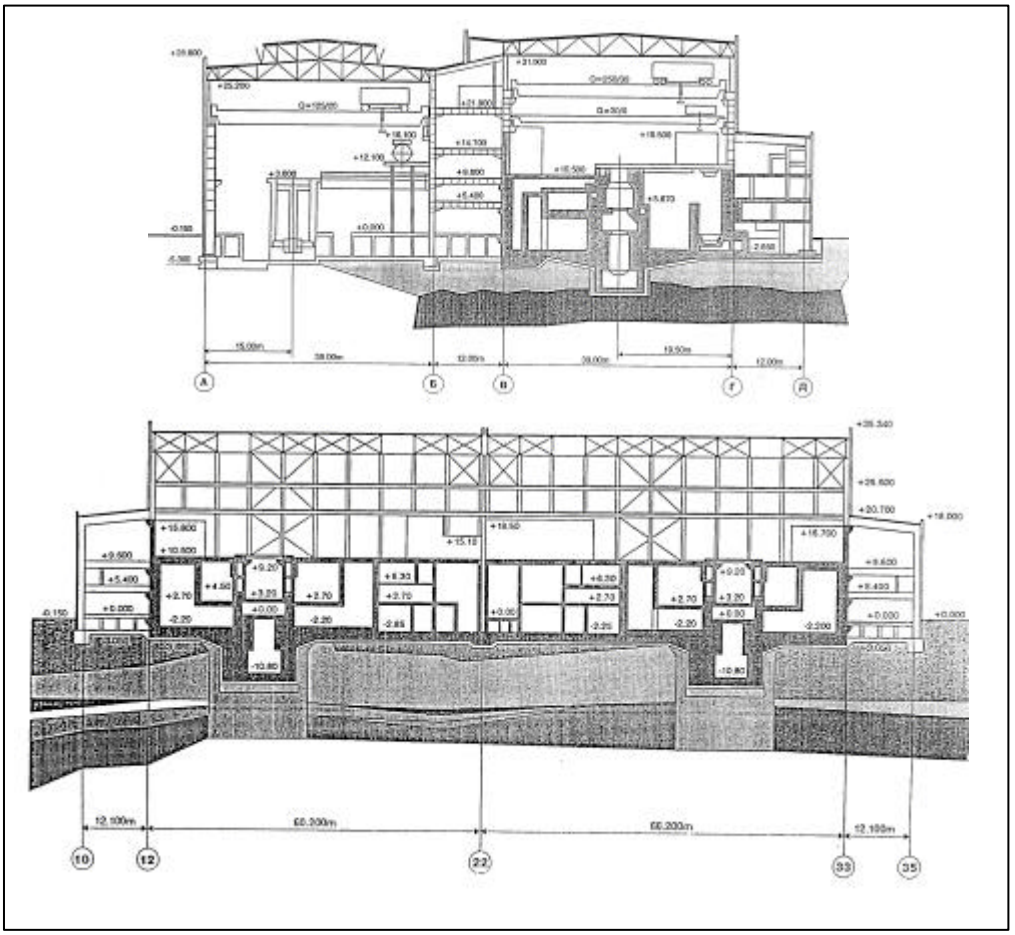


Figure 2 – Transversal and longitudinal cross-sections of main building

The dynamic structural analysis will be carried out accounting for the soil-structure interaction effects. The influence of property variation of soils will be taken into account, by assuming an appropriate variation percentage band. According to the IAEA Technical Guidelines [1], SSI effects can be ignored if the changes in the natural period of the system are less than 10%. Floor Response Spectra will be calculated as a minimum on those locations where systems, and components required to ensure the safe shutdown condition of the plant are positioned. These locations are derived by the SSEL.

The seismic capacity evaluation of buildings and structures will be carried out by using a deterministic approach basing on permissible damping values and inelastic seismic demand of members developed in during IAEA reviews of the Bohunice, Kozloduy and Paks NPP [1]. Material strength will be based, as far as possible, on existing test data. The need for a characterisation survey to assess ageing of structures is being presently evaluated by ANPP in order to rely on actual data of material properties. Which ever are the reference material data, a very little probability that actual strengths are less than those used in the seismic capacity evaluation will be considered. Strength capacity estimation should be based on the ultimate strength approach in accordance with codes for the appropriate materials (ACI [5, 6, 7] / AISC [8] or equivalent national or European codes).

The integrity of non structural elements (not explicitly modelled) and the evaluation of members capacity to withstand loads transferred from relevant mounted-on equipment will be assessed in the scope of the seismic capacity evaluation of safety related System and Components. Analyses methods and assessment

criteria for seismic capacity evaluation of systems and components are outlined in the IAEA Technical Guideline [1].

SAFE SHUTDOWN EQUIPMENT LIST (SSEL) AND PLANT WALKDOWN

A fundamental task of a seismic re-evaluation process of an existing NPP is the definition of the essential Structures, mechanical, electrical, I&C and distribution Systems and Components (SSC) and their function requirements during and after the occurrence of a RLE as well as Plant Walkdowns. Both the activities are aimed at limiting the scope of the plant to be re-evaluated.

The development of SSEL should be based on the main assumption that the plant must be capable to reach the safe shutdown condition and to maintain it in the first 72 hours after the occurrence of the seismic event. This criterion, however has to be assessed specifically for the plant situation, taking into account the specific conditions of systems and cooling requirements as well as availability at the site of equipment and spare parts to repair the damaged systems and components.

The first draft of the SSEL was developed by the Russian Institutes (Gidopress and NIAEP) and reviewed by IAEA in the course of the follow-up review mission performed in June 1999 [9]. Although the draft SSEL is not yet optimised, the final version is expected to reduce up to 90% the selected group of SSCs.

Structures, Systems and Components listed in the SSEL were examined in the plant walkdowns carried out by the plant technical experts in order to evaluate those systems and components that can be eliminated from further analyses, taking into account up-grades carried out in the past years of operation.

The plant walkdowns were focused on:

- equipment characteristics and inherent seismic capabilities,
- anchorage of the equipment,
- load path form the anchorage through the equipment and
- spatial interactions.

The preliminary screening seismic walkdown was conducted in the year 2000 by U.S. DOE and NRC representatives with the aim to screen out systems and components that are obviously robust and require no additional evaluations (disposition category 3). Subsequently a detailed screening walkdown will be performed with the objective to individuate in the final SSEL:

- components whose seismic capacity is uncertain, thus requiring further analysis to determine if a modification is needed (disposition category 2)
- components for which a modification is required (disposition category 1).

In this second phase, limited analytical evaluations (including anchorage capacity, load path and potential spatial interactions) are provided as well as judgements on the modification needed for safety up-grades.

PREPARATION OF THE TECHNICAL SPECIFICATION FOR STRUCTURE ANALYSIS

Technical Specifications for “Completion of Geotechnical Data Collection of the Site and Assistance to NIAEP in Performing Floor Response Spectra (FRS) and Seismic Capacity Evaluation of Buildings and Structures” were prepared in the year 2000 by SOGIN under EU/TACIS program (A1.01/96/I) and have already been approved by ANRA and EU at the beginning of 2001.

The site consists of a basalt matrix, within which layers of softer material–weathered basalt and/or residual soil – are interspersed. According to IAEA recommendations [9] the only missing data are the strain dependent properties of shear modulus and damping for soft residual soil layers and two ways were indicated to derive them:

- to perform laboratory tests or
- by means of scaling of strain dependent parameters on the base of literature survey.

The assistance to NIAEP in evaluating the seismic response and capacity of the NPP Unit 2 will be carried out by means of an independent analysis. Methods and criteria were specified in the Technical Specifications in accordance to IAEA Technical Guidelines [1], also taking as reference the European Utility Requirements (EUR) for LWR Nuclear Power Plant [10]. As part of the assistance task a strong interaction with NIAEP is required in order to compare the results of the two different structural analyses, highlighting the main differences and analysing the sources causing results deviations.

From this reconciliation process a unique set of FRS must be assessed to be used in all the further seismic analyses of systems and components as well as the design of up-grades. Furthermore it must be taken into account that safety improvements have already been performed (e.g. replacement of SG and PRZ safety valves under EU/TACIS program and MSIVs under U.S. DOE assistance program) having considered in-structure seismic inputs at that time available. Then a comparison and reconciliation process between the new FRS and the ones adopted for the already implemented up-grades should also be foreseen.

CONCLUSIONS

The seismic re-evaluation of Medzamor NPP started in 1993 and since then different Organisations have been involved to provide the best safety improvement to the plant. The IAEA Technical Guidelines document, which was adopted by the Armenian Regulatory Body (ANRA) as regulatory document in 1999, provides the general framework, including analysis methods and assessment criteria, within which the seismic re-evaluation program has to be carried out.

To date a complete seismic hazard of the site was carried out and it was finalised in the assessment of Review Level Earthquake to be used in the re-evaluation process. The evaluation of the seismic response of the NPP Unit 2 and consequently the capacity of buildings and structures is presently underway and its completion is envisaged by the end of 2001.

The results of the seismic response analysis together with the final development of the SSEL and plant walkdown evaluations will permit to carry out the seismic capacity assessment of systems and components whose development is envisaged in 2002. It is worth to mention that the availability of the above mentioned results will have an impact also on the implementation of safety related projects currently in progress.

Potential modifications to SSCs will be prioritised for design and implementation, giving considerations to risk/safety, but also to economic and ease of implementation. Detailed criteria on this issue as well as design codes and quality assurance have still to be assessed, basing on an agreement between all involved parties – regulator, operator and donor Institutions.

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THE SEISMIC ASSESSMENT OF BRITISH ENERGY'S NUCLEAR POWER STATIONS AND SOME PRAGMATIC SOLUTIONS TO SEISMIC MODIFICATIONS.

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ABSTRACT

British Energy owns and operates 8 Nuclear Power Stations in the United Kingdom. These include seven Advanced Gas-cooled Reactor (AGR) sites and one Pressurised Water Reactor (PWR) site at Sizewell B.

As part of the site licence conditions the structures, plant and systems have to be maintained throughout their operational life in such a manner as to maintain their fitness for purpose and to carry out the role allocated to them by the reference safety case. A review, referred to as the Periodic Safety Review of the plant, is carried out every 10 years. The original design intent, changes to codes of practices and the effects of any ageing and/or deterioration are considered and any remedial action necessary is identified. The effects of external hazards are considered as part of that review.

Many of the older stations were not designed for, and had never been assessed for the effects of earthquakes. As a consequence, major review work against site-specific seismic hazards has been carried out. In general, the seismic assessment of the plant, systems and structures relies on "seismic walkdown" techniques, seismic qualification databases, similarity arguments, mathematical models and code of practice comparisons. These techniques are applied to the "success paths" set out for the two lines of protection required under British Energy's "Nuclear Safety principles". Where any of the above arguments result in plant modifications, these are implemented on site.

The assessment process is described in this paper and some pragmatic solutions to the retrofitting of restraints to, amongst others, electrical cabinets, pipework, masonry walls, and tanks are discussed. Some novel techniques such as the use of structural adhesives are described in detail.

INTRODUCTION

British Energy owns and operates 8 Nuclear Power Stations in the United Kingdom. These include seven Advanced Gas-cooled Reactors (AGR) sites and one Pressurised Water Reactor (PWR) site at Sizewell B.

Because of the potential risks involved, modern nuclear power stations are among the few structures in Britain that are specifically designed or assessed against the effects of earthquakes. The newer power stations are designed for a postulated seismic event with a return period of 10,000 years, the older stations are assessed as part of the Periodic Safety Review (PSR) required by Site License Condition 15. The PSR considers the assessment of hazards and, in particular, risks presented to the station from a seismic event. As many of the existing safety cases for the older stations did not include a statement on seismic capability, the PSR required that an assessment of all essential plant, systems and structures be carried out and any necessary modifications to improve the seismic capability be implemented. This essentially meant that the functions of Reactor Trip, Shutdown and Post Trip Cooling could be demonstrated to be capable of surviving the defined seismic event and that two lines of reactor protection would be available. It is worth noting here that, although not specifically included in some of the early designs, hazard loadings such as seismic have to be considered. There are inherent and significant margins available to resist the seismic loads due to the substantial construction of modern nuclear power plant structures, plant and systems.

STRATEGY

The strategy adopted for seismic assessment under the PSR required that appropriate plant be designated in two categories:

- Bottom Line Plant and Structures: These are plant and structures required to provide a single line of protection during infrequent, more onerous, seismic events and to provide one of the two lines of protection against lesser frequent events. A full listing of Bottom Line Plant and Structures is set down at the start of the PSR.

- Second Line Plant and Structures: These are plant and structures that provide the second line of protection to ensure an adequate level of redundancy in safety related plant during the more frequent, less onerous, seismic events. A full listing of the Second Line Plant and Structures is also prepared at the start of the PSR.

The 'Second Line' plant is normally assessed against the Principia Mechanica Ltd (PML) design response spectrum appropriate to the ground conditions at the site factored such that the zero period acceleration is 0.1g. The response spectrum representing the 0.1g (PML) input motion is shown in Figure 1.

GENERAL APPROACH

For a typical UK Nuclear Power Plant, the typical assessment approach would be as follows:-

- Determine the probabilistic hazard level and get agreement with the Regulator.
- Investigate the various techniques (time history, response spectrum, static equivalent etc.) for applying the hazard to typical nuclear station structures and establish the most appropriate methods.
- Utilise realistic criteria (stress/strain levels, ductility) that would normally be utilised in assessing the effects of the seismic hazard on typical structures rather than the more onerous new design approach. In particular, the use of internationally agreed 'best practice' in walking down the plant and using the database listing examples of plant's capability to survive real earthquakes would be the main pragmatic approach to minimum qualification.
- Apply the above techniques to a list of essential plant, systems and structures and assess individual seismic capability of each item in turn.
- Only consider modifications if ALARP to do so. (Nuclear Power Stations are legally bound to demonstrate that they have reduced the risk to As Low As Reasonably Practicable (ALARP). The ALARP principle states that unless the expense is in gross disproportion to the risk, the nuclear station must undertake the expense to reduce the risk.)
- Finalise safety case.

SEISMIC DESIGN AND ASSESSMENT STANDARDS

The design philosophy adopted for assessment of the seismic capability of existing plant is based upon the guidance of the SQUG Generic Implementation Procedure (GIP) [1] which utilises experience data available from plant and equipment which has been subjected to actual earthquakes. This approach is supplemented by the Seismic Design Guidelines [2], which provide a guide to the design of retrofit seismic restraints and modifications necessary to achieve formal qualification.

A number of conservative parameters exist within the seismic design methodology and these are sensibly reduced where intrusive and/or large modifications are generated by the analytical process. These may be summarised as follows:

Natural frequency of response for plant items is assumed to coincide with the peak acceleration values from the secondary response spectra. This conservative assumption generally provides upper bound seismic demand values for the item of plant to be assessed/modified. Further calculations may be performed to calculate the lowest frequency of response that can be expected and hence lower acceleration values adopted.

The GIP provides estimates of mass for different classes of nuclear power plant and equipment that constitute the maximum values recorded from inspection and surveys of actual equipment. Based upon experience of internal inspection of electrical cabinets, these values can be shown to overestimate the

actual mass values by a margin of 2 to 3. Actual mass values may be derived from internal inspection, lifting records or identification plates attached to the plant.

A factor of conservatism f_{con}=1.25 is applied to the secondary response acceleration values for the assessment of anchors. This factor is deemed to allow for assumptions inherent within the dynamic modelling techniques. Scope exists to reduce this factor to unity where its application can clearly be shown to be conservative e.g. (1) adoption of a peak acceleration value which is formed by a localised peak within the secondary response spectra or (2) conservatism within the derivation of the secondary response spectra which can be highlighted and quantified.

The Seismic Design Guidelines apply further design load factors to the seismic demand values depending on the design code being considered e.g. steel bracket restraints would invoke a further load factor of 1.20 for BS5950 design implementation. This additional load factor is judged to introduce unnecessary additional margin into the design process and is sensibly reduced to unity to avoid excessive steelwork modifications.

The above areas of conservatism within the design process are investigated where non-standard solutions or extensive/intrusive modifications would otherwise be generated. Furthermore, any inherent seismic withstand capability and robustness of the existing unmodified plant/equipment is generally ignored during the qualification process, i.e. (1) existing anchorage is generally ignored and (2) start-up loads / operational design loads may bound seismic effects for some plant items. Generally, a pragmatic engineering approach is adopted to produce restraints that are practical and safe to install on site.

The GIP provides the most comprehensive guide to the seismic capability of nuclear power plant currently available. The design philosophy adopted is aligned to the guidance provided therein and applies sound engineering judgement based upon standard structural and mechanical principles. Further reference is made to applicable design standards [3] during the design/assessment process and the most appropriate methods of analysis are adopted to produce safe and conservative solutions whilst satisfying the practical resolution of identified shortfalls.

Although a number of plant items on the stations had shortfalls, it was considered that these could be readily addressed through simple modification, or in special cases, through time at risk arguments. Most of the items identified as requiring some modification involved strengthening of structural steelwork and brick walls, improving anchorages to electrical cabinets, and removing excessive movement of pipes, etc., which could result in overstressing or physical damage.

PIPEWORK

The following approach was used to qualify pipework. to resist the input motions resulting from a seismic event. The assessments were carried out for the defined infrequent earthquake hazard level. The infrequent (10^{-4} per annum) seismic input motion was defined as the Uniform Risk Spectra (URS) for a hard site anchored to 0.23g peak freefield ground acceleration (PFFGA) or the URS for a soft site anchored to 0.17g PFFGA.

The assessments utilised a combination of techniques involving analytical and design calculations plus walkdown techniques. The procedures were as follows:

- Appendix B of Nuclear Electric Memorandum TEM/MEM/0096/94, Training Guidelines for New Users of Seismic Walkdown Procedure [4]

- Seismic Design Guidelines , Report Number EPD/AGR/REP/0072/96 (Issue 2) [2]

These were used to carry out the seismic integrity assessment of essential pipework. The assessment methods, as set out in [2] (based around simple structural models of the pipework) and stress acceptance criteria are set out in BS 806 [3], and are known to be overly conservative. All the seismic experience data that is available suggests that smallbore pipework is very robust against seismic displacements and inertial forces. EQE have produced two reports [5] and [6], which detail more realistic screening and acceptance criteria to allow the seismic qualification of a larger proportion of the identified smallbore lines with imposed displacement concerns (seismic anchor movement, SAM) without modification. The approach developed is for smallbore lines specifically (up to 3 inch nominal bore). The EQE reports were subjected to an independent peer review that endorsed the use of the EQE work and deemed it to be a reliable methodology to be used in conjunction with other screening criteria when assessing the adequacy of smallbore pipework.

The following procedure was used for assessing the seismic adequacy of smallbore pipework:

- A list of all essential smallbore pipework that required assessment as part of the seismic safety case was generated.

- Suitably experienced persons carried out a detailed walkdown and seismic review of the essential pipework arrangements and associated equipment. This review recorded if there were any outliers (items that did not meet the screening criteria). Screening guidelines were based upon the USDOE 545 [7] and the SQUG GIP, [1] for pumps, nozzle loads etc.

- Calculations and detailed assessment of pipe arrangements where carried out in accordance with the Seismic Design Guidelines. Associated equipment was assessed using the SQUG GIP.

- The EQE methodology [5,6] was used to address any smallbore lines with imposed displacement concerns.

The findings of the above (including seismic spatial interaction hazards) were summarised in tabular form. Any outstanding shortfalls were listed along with suggested remedial actions for their resolution following which modifications arising were implemented on site.

To calculate a level of reliability it is necessary to consider the loading and resistance functions. The loading and resistance functions are represented by two normal distribution curves and this technique is effectively classical structural reliability theory. As the earthquake loading function was not considered as part of the original design of the stations, the effort was concentrated on the resistance functions and whether or not these were code compliant or had to be modified (e.g. by additional restraints).

In the case of the smallbore pipework, the system was initially code of practice designed for a whole series of loadings e.g. pressure, temperature, self weight, live load, dynamic loads, corrosion and stresses induced by working the metal amongst others. The reliability of such a well designed system constructed to code gives a reliability of around 10^{-5} per demand. Referring to the walkdown process set out above, the first pass used the SDG [2], which is predominately a design code approach, and the parts of the system passing this criteria gave at least 10^{-5} per demand. The next approach was the use of calculations using software such as ADLPIPE and PSA5. As the systems were assessed to be code compliant, then a reliability of at least 10^{-4} per demand could be claimed. This allows for the additional “seismic” stress to be included in the overall resistance functions. On the parts of the system falling outside these two approaches, the screening criteria in References [1] and [7] were used to give more realistic assessment criteria. In

addition, References [5] and [6] were used to address seismic anchor motions. Notwithstanding these assessments, any required modifications were designed to codes of practice and easily give 10^{-5} per demand.

Thus, as the system was rigorously examined on site by experienced walkdown engineers under the procedure set out above, is subject to a maintenance and test regime and has been modified to ensure code compliance, an overall reliability of 10^{-4} per demand can be claimed.

MASONRY WALLS

Masonry walls can pose two types of threat to an essential system:

- Loss of stability and collapse onto essential plant;
- Loss of support for wall-mounted equipment.

Therefore if a wall is considered to have potential to fall onto essential plant an assessment is undertaken on its seismic stability. If the wall is noted as providing support to an essential item of plant, an assessment is undertaken to determine the propensity of the walls to suffer seismically induced cracking. The results of the onset-of-cracking calculations are used to determine whether there may be deleterious effects on the anchorage of wall-mounted plant as a result of the seismic behaviour of the wall.

The BS 5628 [8] flexural assessment method is used to establish the onset of cracking and either the Reserve Energy Method (REM) or the BS 5628 arching assessment can be adopted as a check on the seismic stability of the walls.

The three available assessment techniques can be summarised as follows:

- **Flexural Assessment**

This assessment determines the onset of seismically induced cracking. It is based on the standard BS5628 flexural assessment.

- **Reserve Energy Method Assessment**

The REM has been adopted as a stability check for masonry walls. The restoring moment due to the self-weight of the wall resists the seismic lateral motion on the wall. However, if the seismic displacement is sufficient to move the centre-of-mass a distance of half of the thickness of the wall, then the restoring moment is lost and the wall is said to have lost its stability. Within the assessment it is assumed that the wall has a response frequency coinciding with the peak spectral displacement at the floor level. Therefore the peak spectral displacement is compared to the thickness of the wall. A margin of 2 is required to ensure that the restoring moment is not lost, and conventionally an additional stability margin of 1.75 is also required.

If the peak spectral displacement is greater than that allowed, then a more accurate prediction of the seismic displacement is made from the displacement spectra by assuming the wall behaviour is similar to that of an inverted pendulum using the response frequency calculated in the normal way.

- **Arching Capacity Assessment**

It is recognised that a masonry wall exhibits additional lateral load carrying capacity if it can be shown that it arches within its support frame. An additional stability check is available in accordance with the arching methodology detailed in BS 5628 (Clauses 36.4.4 and 36.8) [8] by calculating the resistance to seismic loads due to the arching of a wall within its frame and an assessment of the arching behaviour of masonry was calculated on this basis. It is imperative, if it is intended to utilise this method, that the walls are inspected in detail to ensure the adequacy of the top-edge mortar joint and to ensure that the structural frame is sufficiently robust to accommodate the arching loads.

ELECTRICAL CABINETS

The principles given below were used for designing seismic restraints to electrical cabinets at our AGR Power Stations taking account of the recommendations of EPRI report NP-5228-SL [9]. Prior to commencing any design work, the designer should review the anchorage assessment calculations, or the SQUG Screening Evaluation Work Sheets (SEWS), to determine the nature of the seismic shortfall. In some instances, the tightening of existing bolts may be all that is required. However, care should be taken when checking the tightness of shell-type expansion anchors, to ensure the check is not simply tightening the shell against the bottom of the equipment base (Ref. GIP Figure 4-4 [7]).

If there is insufficient information detailed on the SEWS then it will be necessary to inspect the cabinet on site to:

- take dimensions and photographs.
- note the space and access restrictions.
- note any special factors e.g. large cables; other fixings.
- note the existing base fixings.
- note any interconnecting bolts.
- determine the best connection points to the cabinet. Base restraint is the preferred option, however if this is not possible, head restraint can be provided via eye-bolts on top of the cabinet, or in certain circumstances by the use of structural adhesive.
- consider the restraint options

The general methodology for the calculation of design forces is given in EPRI report NP-5228-SL, Vol. 1, Section 3 [9].

The stability of the cabinet should be checked in both directions and if the stability of the cabinet is marginal then due cognisance can be taken of existing anchor bolts and attachments in an attempt to resolve the shortfall.

In cases where base restraint only is being considered the tensile loads developed in the equipment base should be calculated and compared with anchor capacity, noting that no uplift of the cabinet base is permitted. If head restraint is being considered, the reaction force required at the head of the cabinet to ensure that no tension is developed under the equipment base should be calculated. In addition a check should be carried out to ensure that there is sufficient resistance provided at the base to prevent sliding.

If a restraint is deemed necessary then the best seismic restraint option must be determined. For a floor mounted cabinet, the best solutions in order of preference generally are:

- **Type A Base Restraint**: If the access doors/panels permit, a base restraint (designed to resist overturning and sliding) composed of external angle brackets secured with adhesive to the cabinet and bolted to the floor may be used. If the cabinets are sitting on base frames, it must be ensured that the connection from the base to the cabinet is strong enough if the brackets are only attached to the base frames, otherwise extended brackets may be required.

- **Type B Head Restraint**: This option comprises a raking strut bolted to the cabinet head (utilising an existing lifting eye-bolt) and bolted to the soffit of the floor slab or beams above. In all cases the proposed head restraint must avoid existing services. In addition floor plates bolted to the floor should be provided to carry sliding loads, if the existing cabinet anchorage bolts are insufficient to take the shear load. If the slenderness ratio of the proposed struts is too large then the use of opposing ties should be considered instead.

- **Type C Head Restraint**: This option comprises struts bolted to the cabinet head (utilising an existing lifting eye-bolt) and to existing structural columns. In all cases the proposed head restraint must avoid existing services. In addition floor plates bolted to the floor should be provided to carry sliding loads, if the existing cabinet anchorage bolts are insufficient to take the shear load. It should be noted that if the existing lifting eye-bolt is inaccessible then structural adhesives may be utilised to connect the strut to the cabinet head providing that the strut is horizontal.

- **Type D Head Restraint**: This option is comprises struts bolted to the cabinet head and to new or existing secondary members spanning between main structural columns. In all cases the proposed head restraint must avoid existing services. In addition floor plates bolted to floor should be provided to carry sliding loads if existing cabinet anchorage bolts are insufficient to take the shear load. It should be noted that if the existing lifting eye-bolt is inaccessible then structural adhesives may be utilised in the connection of the strut to the cabinet head providing that the strut is horizontal.

The appropriate seismic input accelerations are determined for the item of equipment in each of the three directions of motion. For the base restraint option, it is assumed that the cabinet is flexible (GIP Appendix C.1[1]) and responds at the peak of the appropriate response spectra. For head restraint options the support frame will inevitably increase the natural frequency. This revised frequency can be calculated and used to obtain the appropriate acceleration level for the system or alternatively it can be conservatively assumed that the system continues to respond at the peak of the response spectra.

The seismic restraint of electrical cabinets was designed to resist the peak of the SQUG GIP Reference Spectrum (1.2g [1]). However, in cases where detailing difficulties arose as a consequence of adopting the GIP spectrum, the anchorages and/or restraints were designed to resist the peak of the appropriate secondary response spectra derived for the floor level under consideration. The secondary response spectra were derived from the 0.14g peak free field horizontal ground acceleration Principia Mechanica Ltd. (PML) hard site response spectrum, which essentially enveloped the 10^{-4} per annum uniform risk spectra.

The seismic inertia loads were calculated for each of the three directions of motion using the equivalent static technique:

Inertia Load = factor of conservatism $\times$ equivalent static factor $\times$ acceleration $\times$ mass of equipment

The GIP Table 4-3 [1] shows that a factor of conservatism should be applied to take account of the type of in-structure secondary response spectra used. A factor of 1.25 was considered to be appropriate but it should be noted that if the GIP Reference Spectrum is adopted then a factor of conservatism of 1.0 should be used. The EPRI report [9] shows that an equivalent static factor of 1.0 is adequately conservative for determining the base shear and overturning moment of base-supported equipment. For cabinets with top supports, an equivalent static factor of 1.5 should be used to account for forces from higher order modes. The mass of the equipment is derived from the densities of various equipment types given in the GIP, Rev 2A, Table C.1-1 [1].

The seismic inertia load is applied at the centre of gravity of the equipment and are distributed to each of the anchors by calculating the following force components from each direction of motion:

- Anchor shear due to horizontal seismic inertia loads
- Anchor shear due to torsional seismic inertia loads
- Anchor pullout due to seismic overturning moment (with an appropriately assumed location of the overturning axis).
- Anchor pullout due to vertical seismic inertia loads

The anchor shear due to seismic motion is the vector sum of the direct shear and the shear due to torsion.

Sections 6 to 16 of the EPRI report [9] provide recommendations for the location of the overturning axis for various classes of equipment. The general approach is to locate the axis conservatively at the equipment base centreline when the base framing is judged to be flexible. Otherwise the overturning axis is located at the edge of the equipment base. Guidance on base flexibility is given in the GIP - Section 4.4.1[1]. (It should be noted that all electrical cabinets are deemed, by the GIP [1], to have a flexible base when considering overturning.)

Once the location of the overturning axis is determined, the anchors located in the tension zone of the equipment base are identified and their centroid calculated. The centroid of the compression zone is calculated from the perimeter length of the equipment item in the compression zone and a linear distribution of the compression stresses in the equipment base as a function of the distance from the overturning axis.

The pullout load is calculated for each anchor located in the tension zone so as to balance the imposed overturning moment between the tension and compression zones. Since the anchors may not be symmetrically located relative to equipment centrelines, each of the two directions of motion parallel to the plane of the anchor group is evaluated in both its positive and its negative direction.

The analysis method described above calculates anchor loads due to each of the three directions of earthquake motion. The co-linear load components on each anchor due to each input motion are then combined using the SRSS method. The maximum net tension on each anchor is determined for each directional combination by subtracting the compression due to dead load of the equipment from the seismic tension value. Frictional resistance is excluded from the analysis as an unquantifiable conservatism.

Expansion anchors are generally used to fix into structural concrete. It is important to ensure that the anchors are of sufficient length to guarantee that the load is transferred through the floor screed and into the structural concrete below and that the chosen anchor type should be of a comparable type to those listed in Appendix C of the GIP[1].

The design of the anchors should be carried out using the method laid out in the GIP Appendix C. This method includes reduction factors that take cognisance of anchor type, anchor edge distance, anchor spacing, embedment length, concrete strength, cracked concrete and the presence of essential relays. All of these factors must be used in the design of the anchors. A further check on the shear / tension interaction in an anchor is made using the Bilinear formulation (refer to GIP clause C.2.11 for expansion anchors).

USE OF STRUCTURAL ADHESIVES

Structural adhesives may be used when fixing steel brackets to the cabinets providing that the joint is designed such that the adhesive is required to resist only shear forces between the two steel surfaces. Tensile loading should be avoided in all cases, as any deformation of the adherents would result in undesirable peeling forces being set up around the edges of the joint.

Large factors of safety are adopted for the design of adhesive connections (a margin of 5 is applied against the ultimate shear resistance of the material to allow for creep and fatigue effects). It is widely accepted that the adhesive/adherent combination does not transmit the applied loading to the central area of the joint and to account for this, an effective design area of bond equal to 50% of the actual bonded area is adopted resulting in an effective margin of 10. The bonded surfaces are designed to withstand shear action only and the significant tensile capability is conservatively ignored within the design procedure.

The strength of adhesively bonded joints is largely dependent on both the quality of surface preparation and the method adopted when applying the adhesive. Guidance on these matters is given below:

Surface Preparation

Although the adhesive is generally tolerant of surface contamination, its final strength can be adversely affected by type and amount of contamination present. Contamination can take the form of paint work, galvanising, oil or dust. In order to achieve a satisfactory level of confidence in the bonded joint, it is therefore important that both adherents are free of contamination prior to making the joint. The following steps should be undertaken in order to ensure that the level of contaminants remaining on the adherents is limited to acceptable levels:

- Prepare both surfaces to be bonded using a portable grit blaster complete with vacuum hood to ensure grit particles are not deposited throughout the working area (a hand grinder or hand abrasion may be considered at locations where the use of a portable grit blaster is not appropriate). When completed, the grit blasting should expose the bare steel of both adherents with all paint, galvanising etc. removed.

- Following grit blasting both surfaces should be carefully cleaned to remove contamination. All solvent should be allowed to evaporate completely prior to the application of the adhesive.

Following the completion of the surface preparation it is important that the joint is made in a timely manner to ensure that no contamination is allowed to come into contact with the surfaces to be bonded (eg. by human touch).

Application of Adhesive

The adhesive should be applied to the full contact area between the cabinet and the restraint. All adhesives should be applied in strict accordance with the manufacturer's recommendations and safety data sheets. Following dispensing the adhesive through the mixing nozzle, the two bonded surfaces must be brought together within the specified time. After adhesive application and joint assembly, a consistent clamping or propping force should be applied immediately after the joint is made and this force must not be removed until a minimum specified time (from closure of the joint) has elapsed.

It is recommended that the adhesive is applied in the following manner:

- Prepare the contact surfaces as described under Surface Preparation above.

- Apply adhesive to the cabinet contact surface of the angle bracket as a bead using the supplied mixing nozzle. The bead of adhesive should be applied in a wave form ensuring that, when compressed, as the joint is made, no air is entrapped and that the adhesive is spread over the full contact area. This will be demonstrated if a small amount of adhesive is exuded from all sides. Care should be taken to avoid the adhesive being applied in closed loops thus trapping air within the joint.

- Ensure that the joint is effectively clamped or propped for a minimum specified time. Do not apply a working load to the adhesive joint until 24 hours have elapsed.

- In the case of base restraint brackets it is also necessary to provide an effective horizontal bed of adhesive to guarantee that no gaps are present between the bracket and the floor slab leading to additional forces being induced into the adhesive joint when tightening up the anchor bolts into the floor slab. Care should be taken to ensure that adhesive is applied over the full base of the bracket, especially around the bolts, and not just around the perimeter.

Quality Control

In order to the ensure that each adhesively bonded joint is of a sufficient standard, the following procedures should be put in place to guarantee the quality of the joint:

- All prepared surfaces must be examined by an Approved Person, who will then supervise the final cleaning and the complete application of the adhesive.

- The Approved Person must then sign a pro forma check sheet to indicate their satisfaction or otherwise with the work carried out in terms of surface preparation and adhesive application.

The Approved Person should be a named individual possessing adequate training and experience in the use of structural adhesives and whose appointment has been approved in writing.

Testing

At suitable intervals, sample joints must be prepared using the adhesive used on site and steel plates with a specified overlap. The procedures used for the site installation should be strictly adhered to when bonding the two test plates. A standard timber jig should be prepared to ensure that the two test strips are correctly aligned and overlapped, and that a standard compression is applied as the joint sets. The test pieces should then be marked with a unique numbering system, enabling the location where the particular batch of adhesive was applied to be easily identified. One of the samples should then be sent, as soon as is practicable, to an approved laboratory and tested in accordance with the American Society of Testing and Materials (ASTM), test No.D 1002-72, to establish the shear capacity of the joint. The test results should be carefully examined and the other sample should be stored, under in-service conditions, for future testing.

CONSIDERATIONS FOR RESTRAINT DETAILING

Connection to the Cabinet

The load path from the centre of gravity of the equipment to the restraint or anchorage must be strong and not excessively flexible. Restraints should be located as close as possible to the main structural path so that prying actions are avoided. In general:

- It must be ensured that any adhesively fixed base restraint brackets are provided with an effective horizontal bed of adhesive to guarantee that no gaps are present between the bracket and the floor slab which could lead to additional forces being induced into the joint when tightening up the anchor bolts into the floor slab.

- Base restraint brackets must be stiff enough that forces are transferred from the cabinet frame to the anchor without excessive bending deformation in the bracket.

- Where the cabinets are adequately fixed to each other, the restraints should be detailed on either every second or third cubicle as appropriate. If the cubicles are currently not adequately fixed to each other, the required interconnection, if practicable, should be achieved using a flat plate and adhesive on top of the cabinet.

- If possible head restraints should be connected to the existing lifting points. If this is not possible then an adhesive fixing to the corner of the cabinet, on the top surface, should be adopted.

- Restraints (both head & base) that are to be fixed to the cabinet using a structural adhesive should be located on cabinet corners at intersections of cabinet framing members such that the adhesive is required to resist only shear forces.

- Bolted connections with serrated washers should be used to connect restraints to cabinet frames.

- All steelwork should be designed to a relevant design code (i.e. design to yield stress of the steel with a load factor of 1.0 for seismic loading and appropriate deflection limits).

Connection to the Structure

The following general principles should be used when connecting to concrete:

- **Floor Slab Connections**

  Individual fixings are preferred but before determining the exact position of the restraint on the cabinet and the measurement from the cabinet to the bolt centre a cover meter survey is to be done on the concrete surface to map the position of the reinforcement in the fixing area. The type of restraint and it's position is to be adjusted to miss all reinforcement bars along the face of the cabinet. Should this not be possible, clearance must be obtained to cut reinforcement. If the anchor bolt position is forced to move out from the face of the cabinet in order to miss all reinforcement bars, the designer should ensure that the stiffness of the restraint angle is increased accordingly. In addition it is important to ensure that the anchors are of sufficient length to guarantee that the load is transferred through the floor screed and into the structural concrete below.

- **Column & Beam Connections**

  Before determining the exact position of the restraints, a cover meter survey should be carried out on the concrete surface to map the position of reinforcement bars in the restraint fixing area. The restraint position is then adjusted to miss all reinforcement bars.

The design of expansion anchors applies a factor of safety of 3 over the characteristic loads stated within the manufacturer's guidance. This margin typically exceeds the level of reduction applied to the anchors to derive a safe working load value.

CONCLUSIONS

The above outlines the process of seismic qualification adopted by British Energy for Nuclear Power Plants and sets out how unnecessary onerous criteria can be eliminated or reduced to provide a more pragmatic solution to modifications, without compromising overall safety. The key message here is that there are inherent and significant margins available to resist the seismic loads due to the substantial construction of modern nuclear power plant structures, plant and systems.

The use of structural adhesives is recommended for restraining electrical cabinets as this reduces intrusive working and is relatively easy and cheap to install.

A series of photographs of modifications carried out on British Energy's Power stations is attached to the slide presentation accompanying this paper.

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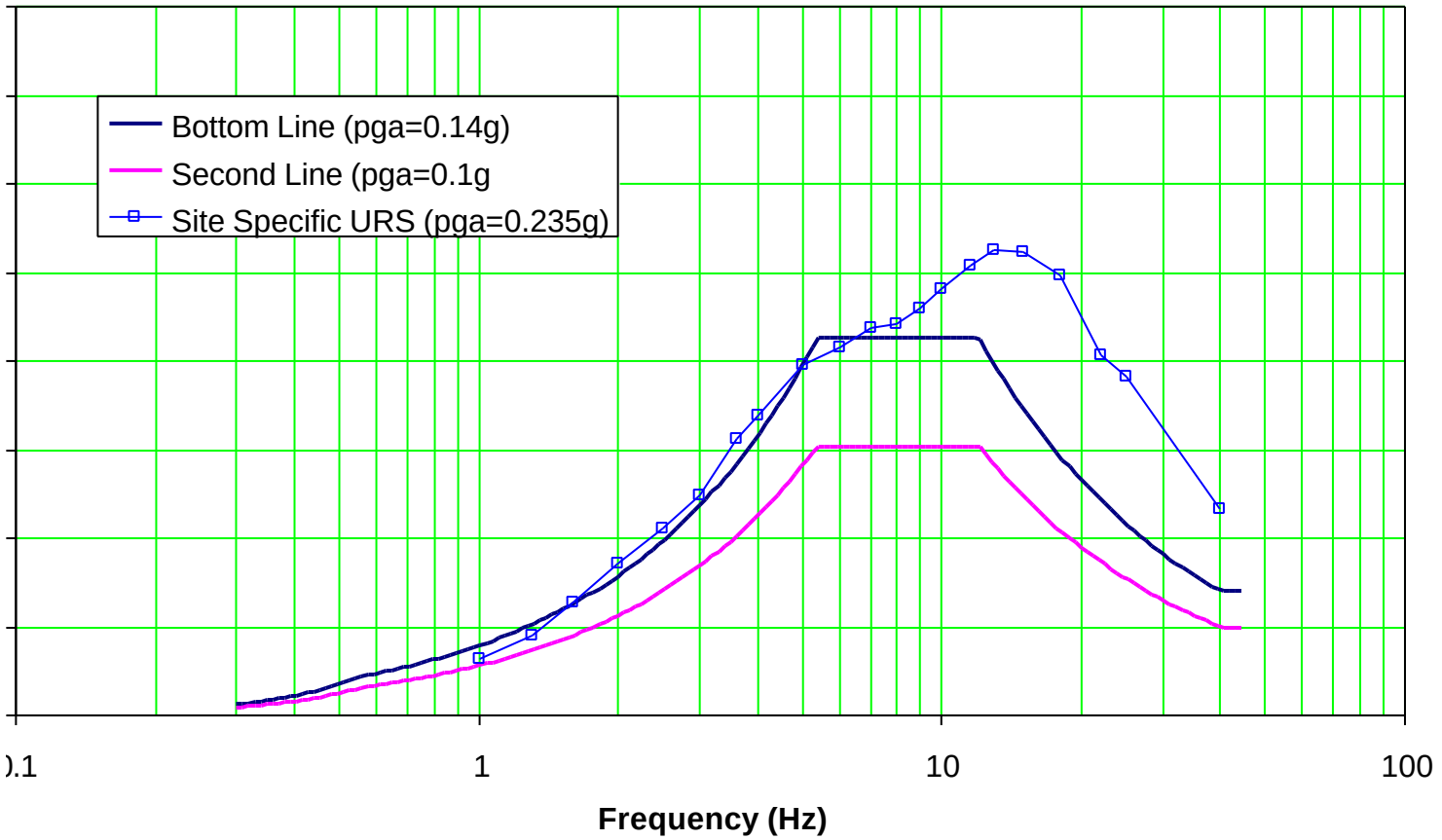
Figure 1



Hinkley Point B Ground Motion Specification

Horizontal, 5% damping

Intercomparison of analysis methods for seismically isolated nuclear structures

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1. INTRODUCTION

The International Atomic Energy Agency have sponsored a Co-ordinated Research Programme aimed at establishing the reliability of analytical methods applied to predicting the behaviour of individual isolation devices and the response of base-isolated structures to earthquake inputs. The Programme was set up following the good performance of base-isolated buildings in the Northridge, California and Kobe earthquakes. The application of this technology to nuclear plants and other related facilities would offer the advantage that standard designs may be safely used in areas with a seismic risk. The technology may also provide a means of seismically upgrading nuclear facilities. Design analyses applied to such critical structures need to be firmly established, and the Programme provided a valuable tool in assessing their reliability.

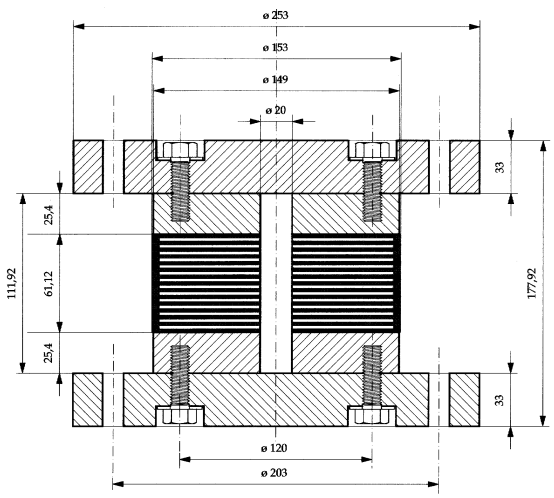
The paper outlines the scope of the Programme, and then summarises the two main aspects of the work. First the use of finite element analysis (FEA) to predict the force-deformation characteristics of laminated rubber-steel isolators is described and the predictions compared with measured behaviour. Both high-damping rubber and lead-core rubber bearings are considered. The second aspect is the prediction of the response to earthquake ground motions of structures isolated by elastomeric bearings. The structures considered are a rigid mass, a steel frame and a storage pool. The accuracy of the predicted behaviours are discussed, and cases highlighted where further development or evaluation of analytical methods is desirable. The results of individual partners contributions to the programme have been reported elsewhere [1,2].

2. Seismic Isolation and Elastomeric Isolators

Isolation of structures from horizontal ground motions is gradually becoming a more common method of providing protection from earthquake damage. In contrast to conventional technology, seismic isolation not only upgrades the earthquake resistance of a structure, but also offers the possibility of protecting the contents and secondary structural features of a building or plant because seismic forces transmitted to the

structure are reduced. The operability and safety of plant can thus be enhanced. The isolation system functions principally not by absorbing the energy of the ground motion but by providing an interface able to reflect the earthquake energy back into the ground. The natural frequency of the structure mounted on the isolators (typically 0.5Hz) is made below the frequencies of strong ground shaking. Damping is needed to limit the displacement of the isolators and control any response at the isolation frequency. During the Northridge earthquake in California (January 17, 1994) and the earthquake which struck the Kobe region of Japan (January 17, 1995) seismically-isolated buildings performed well. For instance, a three-storey laboratory building owned by Matsumuri-Gumi Corporation and mounted on high damping rubber bearings was subjected to a peak ground acceleration of 0.278g during the Kobe earthquake. Immediately above the isolators the peak acceleration in the structure was only 0.151g, and at the roof 0.202g. The corresponding acceleration at the roof of an adjacent office block was 0.985g, almost five times higher.

High damping rubber bearings (HDRB) or lead rubber bearings (LRB) provide a simple and economical isolation system They possess the low horizontal stiffness needed and are capable of safely withstanding the large horizontal displacements imposed on them during an earthquake. The need for additional dampers is avoided. In the HDRBs damping is incorporated into the rubber compound. For the LRBs the damping is provided by a cylinder of lead within the rubber bearing. A diagrammatic section of an HDRB is seen in figure 1. Examples of the use of elastomeric isolators to protect nuclear-related facilities from earthquakes exist within the UK. One such is a day-docking facility for nuclear submarines in Scotland.



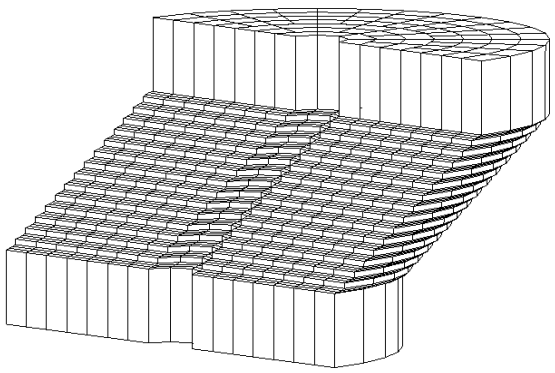


Figure 1: Sketch of the 1:8 scale prototype of the ALMR high damping isolation bearing manufactured by ALGA and tested by EERC

Figure 2: Deformed FEM of a bolted ALMR HDRB during a compression (44kN) and 150% shear strain test

3. SCOPE OF RESEARCH PROGRAMME

3.1 Isolators Behaviour

The proper functioning of isolation systems requires bearings with certain characteristics. The principal ones are:

- horizontal stiffness, K_H
- vertical load capacity
- horizontal displacement capacity, d_{max}
- damping

The isolation frequency is determined by the horizontal stiffness and the gravity load supported. The ability to predict K_H and d_{max}reliably by FEA would mean that the isolator design could be verified with a reasonable degree of confidence before the isolators are submitted to a prototype test programme.

Several isolators were analysed within the Programme by finite element methods to see how well the FEA predicted their force-deformation. The predictions were compared with tests on prototype isolators. Those analysed included:

LRB manufactured and tested in Japan
HDRB manufactured and tested in Italy
HDRB manufactured in Italy and tested in Japan
HDRB manufactured in Korea

In addition to bearing test results, characterisation data for the rubber used in the manufacture was provided; the latter is required by the FE programmes.

Within the Programme, two other types of isolator:

- low-damping rubber bearing
- pneumatic 3D isolation device

were also analysed, but are not considered here. As an adjunct to the FEA of isolators, a benchmark problem – the torsion of a rubber cylinder – was chosen to assess how well the results obtained by the FE solvers compared with those based on an exact analytical solution.

3.2 Response of structures

The design of isolation systems for critical structures obviously requires confidence in the methods used to predict the responses of the structure and isolation system to earthquake inputs. Factors involved include the adequacy of:

- model of dynamic behaviours of isolators
- model of structure

By effectively reducing the seismic input to the structure, and, indeed, allowing the possibility that the response can be kept within the elastic range, the use of isolation may be expected to lead to more reliable analyses.

Within the Programme the response of the following seismically isolated structures was investigated:

- rigid mass

- steel-frame structure (MISS)
- spent fuel pool
- full-scale section of VVER-640 reactor building

The last structure, analysed with the 3-D pneumatic isolation system will not be discussed here.

4. Finite Element prediction of isolator behaviour

The FEA concentrated on prediction of:

- vertical force-deflection behaviour
- horizontal force-deflection behaviour combined with gravity load

Aspects investigated included:

- accuracy of axisymmetric model versus fully 3-D analysis
- accuracy of analysis based on single rubber layer
- influence of mesh density
- effect of different types of material model for rubber
- influence of finite compressibility of rubber

High Damping Rubber Bearings

The FEA predictions were concerned with the quasi-static force-deflection behaviour and did not attempt to predict the dynamic characteristics. The material models used to characterise the rubber stress-strain behaviour were:

- Rivlin polynomial series based on strain invariants
- Ogden model
- Seki model

Overall the predicted behaviour agreed reasonably well with the test results. As an example, results from a 3-D analysis of the HDRB bearing shown in Figure 1 is considered. The deformed mesh is illustrated in Figure 2. The predicted horizontal force-deflection behaviour whilst the bearing is also subjected to vertical load is given in Figure 3 along with the force-deflection hysteresis loop for the maximum shear strain analysed (150%). As stated before the FEA is aimed only at predicting the quasi-static stiffness. The close agreement between the maximum load in the hysteresis loop and the load calculated for the maximum strain within the loop establish the capability of the FEA.

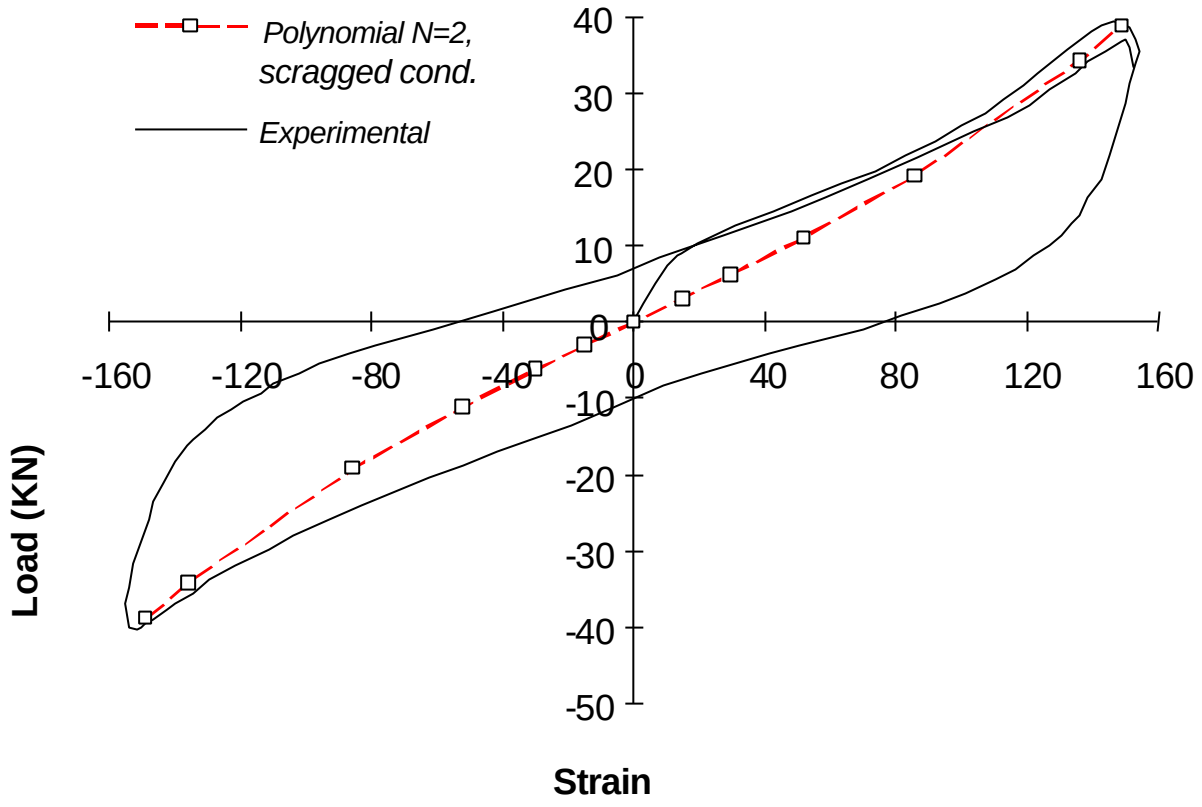


Figure 3: Comparison between measured and calculated horizontal stiffness of an ALMR HDR bearing (1:8 scale, diameter=146 mm, H=61 mm, G=1.4 MPa, bolts attachment system) during a combined compression (44 kN) and 150% shear strain test performed at EERC

For analysis up to moderate shear strains, it was found that the results from an axisymmetric model or a single rubber layer give reasonable results with consequent substantial savings in computing time. For prediction of the vertical stiffness, it is necessary to include the finite compressibility of rubber in the material characterization. Modelling each rubber lamination with a single layer of element was acceptable in predicting the horizontal stiffness, but at least two layers were needed for the vertical stiffness. Provided the rubber material properties data covered a sufficient range of strains and accounted for compressibility, none of the material models chosen to fit the data gave better results overall. The agreement at large rubber shear strains (>200%) even with 3D models was less satisfactory. Moreover, the vertical force-deflection behaviour under imposed shear strains was not well predicted, being sensitive to terms in th material model that are not fitted robustly by the normal characterisation data.

Lead Rubber Bearings

In addition to the factors that need to be taken into account for HDRB, the LRB also require consideration of the behaviour of the lead and how to treat the lead-rubber interface. The horizontal force-deflection behaviour was predicted quite well up to large (400%) rubber shear strains. The vertical force-deflection analysis (for zero shear) also agreed well with test data. A comparison between FEA calculations and observations for the horizontal behaviour is shown for an LRB in Figure 4. Because the hysteresis derives from the yield of the lead, the quasi-static force-deflection curve should be close to the hysteresis loop except at very small strain; this is seen to be the case.

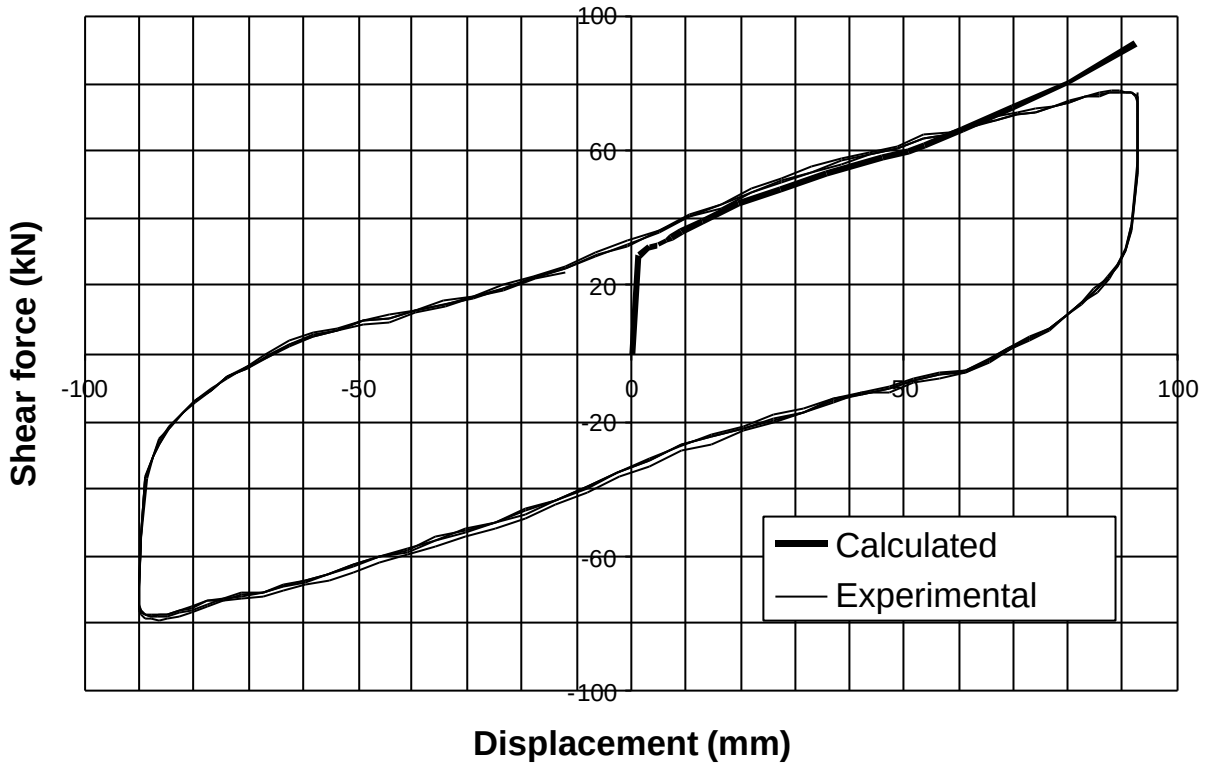


Figure 4: Comparison between the measured and FEA calculated shear force-displacement behaviour for LRB. Diameter 280mm. Lead Plug diameter 70mm. Total Rubber thickness 46mm

With the analyses of the LRB, it was concluded that:

- the lead may be modelled as elastic – perfectly plastic
- friction between lead plug and rubber may be ignored
- axisymmetric FE models work well
- single rubber layer models predict horizontal stiffness reliably

Benchmark Problem

Before the analysis of critical components it is desirable that the FE solvers are validated by benchmark problems. Within the Programme one has been identified and the results from two FE programmes, MARC and ABAQUS compared with analytical solutions. The problem is the torsion of a cylinder for which Rivlin [3] has given a solution.

The FEA determined the couple to deform the cylinder and the axial load required to keep the length constant during torsion. Results for the couple within 5% of the analytical solution, and for the axial load within 10% could be obtained without resort to a very fine mesh provided certain types of element were avoided. The reduced integration solid element in MARC gave relatively poor results.

5. Response of isolated structures

5.1 Simplified model of HDRB and LRB

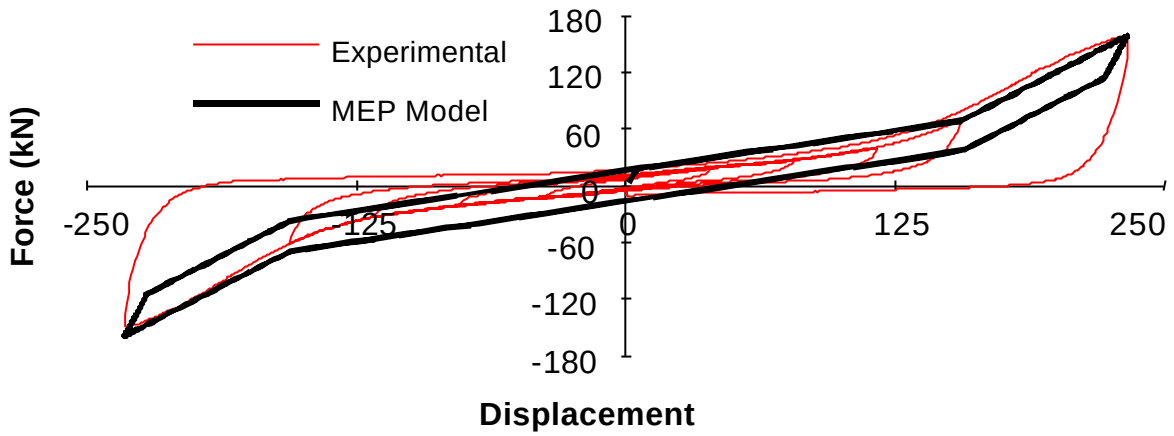


Figure 5: The hysteresis loop of HDRB fitted by MEP simplified model with a single elasto-plastic element

The determination of the response of isolated structures requires a simplified model of the dynamic horizontal force-deformation characteristics of the isolators. The aim here is not to predict the isolator behaviour as with the FEA, but simply to fit the test data. The model has to take account of the damping provided by the isolator, and ideally its non-linear deformation behaviour. One type consists of an elastic spring (linear or multi-linear) in combination with a dashpot element. A more realistic model is provided by an elastic spring, in parallel with one or more elasto-plastic elements to model the damping. The ability of such a Multi-linear Elasto-plastic Model (MEP) with a single elasto-plastic element to fit large shear strain hysteresis loops for an HDRB is shown in Figure 5. It is apparent that the hysteresis at large strain is underestimated. More elasto-plastic elements would improve the fit; such a model has been developed and implemented within the ABAQUS code by ENEL. It has been further refined to take account of the stiffening of HDRBs seen at large shear strains. The good fit obtained, even into the region of stiffening behaviour, to observed shear deformation hysteresis loops for an HDRB (rubber shear modulus, G = 0.8MPa) is seen in Figure 6. The model is equally well applied to LRBs.

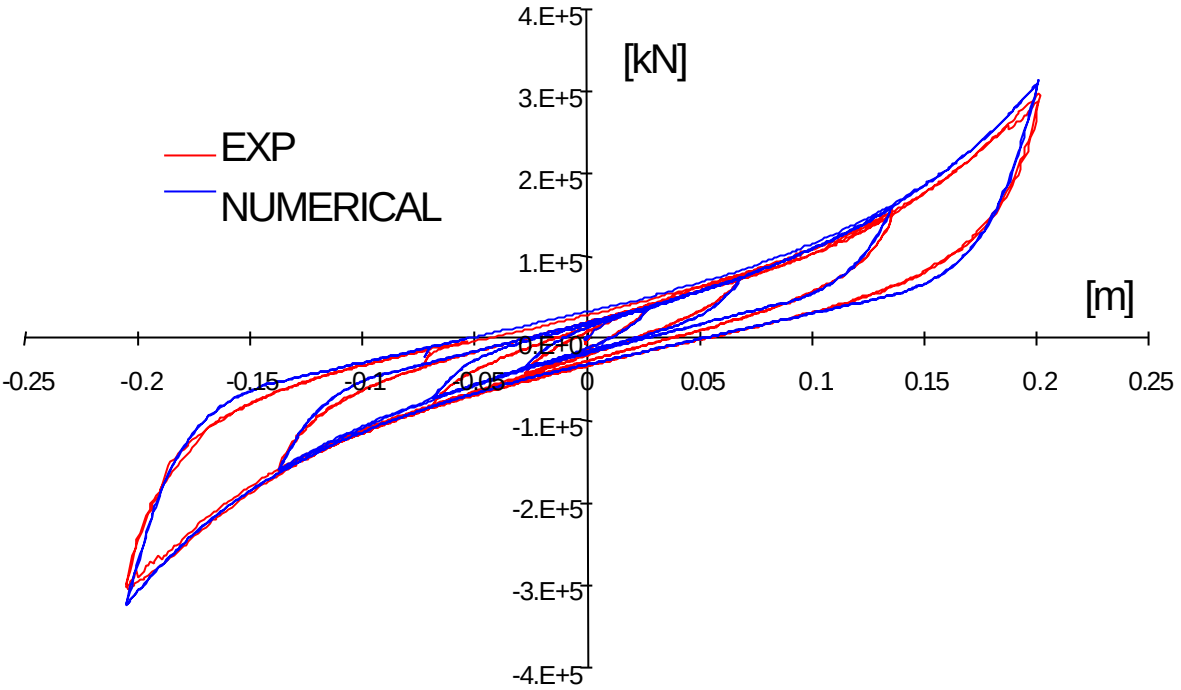


Figure 6: Experimental and numerical hysteresis loops for a HDRB (G = 0.8 MPa).

5.2 Rigid Mass

This was tested at CRIEPI in 1989. The structure consisted of a concrete frame of 178kN weight and size 3x2.1x2.8 (height)m. It was isolated by 8 LRBs.

Prediction of an acceleration time-history using the refined multi-element elasto-plastic model is compared in Figure 7 with the observed history. The fit is seen to be very good. The predictions generally agreed well with observed parameters. The exception was the response to beyond design level earthquakes for which the large rotational motion of the mock-up places isolators in tension; the predictions became sensitive to the vertical stiffness chosen for the isolators.

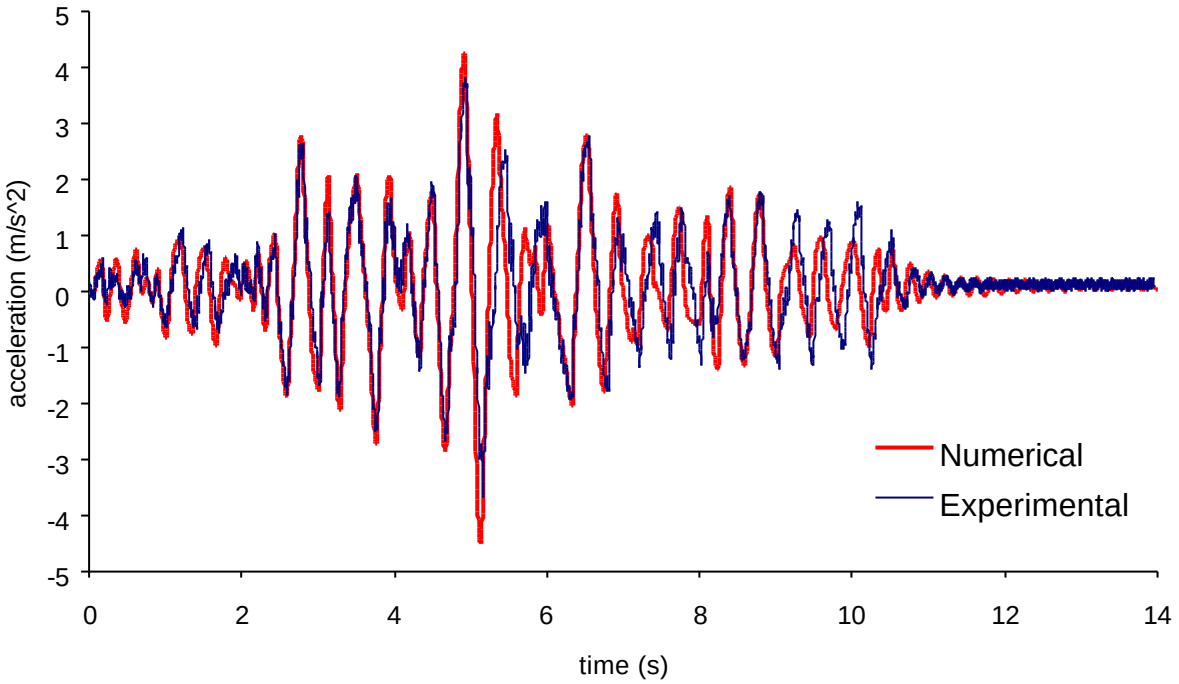


Figure 7: Experimental and calculated acceleration-time histories for rigid mass isolated by LRBs subjected to the design earthquake record.

5.3 Steel frame structure (MISS)

MISS is a steel frame structure mock-up with a rectangular base of 2.1 m x 3.3 m, and four storeys, with an interstorey distance either of 0.9 m or of 1.1 m. It can support up to 20 concrete masses, each weighting 13 kN. The frequency of the structure can be chosen over quite a large range, depending on the interstorey distance and the number of masses used and their disposition. It has been tested on the shaking-table isolated by 6 HDRB (125 mm diameter, 30 mm total rubber height) fabricated with a soft rubber compound (G = 0.4 MPa) and attached by bolts and dowel.

The observations of bearing displacement whilst MISS was subjected to the 1981 Calitri ground-motion record are compared (Figure 8) with predictions using the refined model mentioned above. The agreement is seen to be good.

Generally the response of the isolated structure was predicted well except for the high frequency content. The calculation of the response of the fixed-base structure showed more significant discrepancies.

5.4 Spent fuel pool

the pool mock-up as tested on the shaking-table was of size 2.2x1.15x0.9m and was mounted on 4 isolators. During excitation with the 3 components of the El Centro record, sloshing amplitude hydrodynamic pressure and accelerations were recorded. Modelling of the non-linear sloshing

behaviour of the liquid with either ABAQUS/Standard or ABAQUS Explicit has not so far provided satisfactory results.

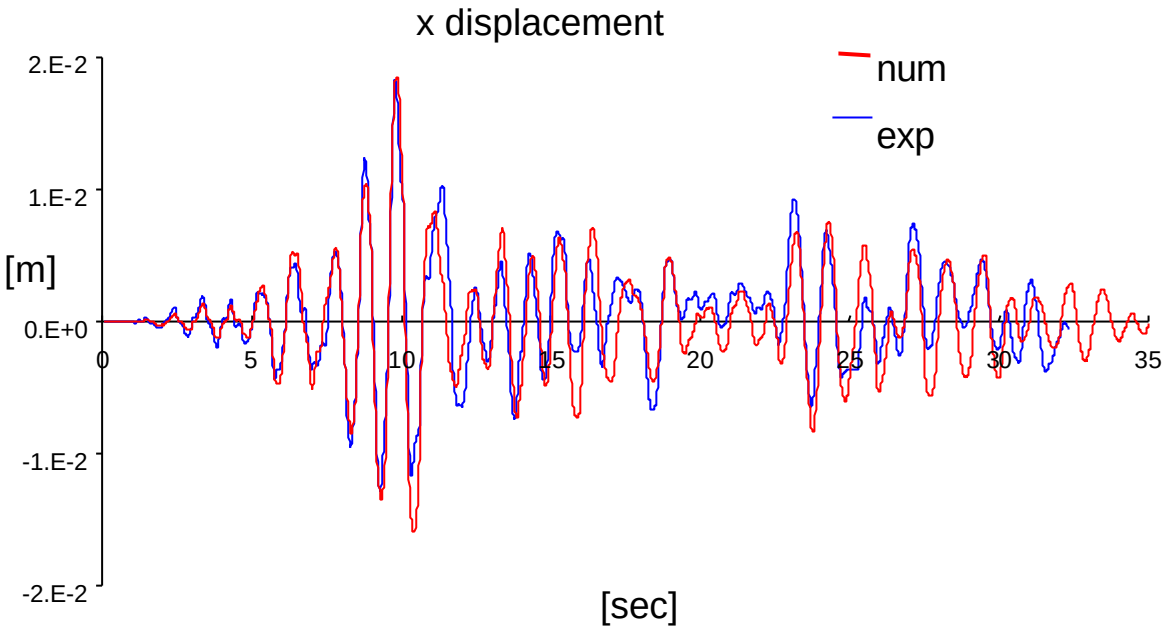


Figure 8: Bearing displacement (x component)-time history for MISS steel frame mock-up isolated on HDRBs and subjected to the 1981 Calitri record

6. Conclusions

The Co-operative Research Programme has shown that predicting the force-deflection characteristics of isolators, and calculating the response of isolated structures can often be done with good results. Areas requiring further work have been identified.

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2. Proceedings of the International Post-SMiRT Conference Seminar, “Seismic Isolation, Passive Energy Dissipation and Active Control of Vibrations of Structures”, Taormina, Italy, August 1997, GLIS

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Seismic reevaluation of PHENIX reactor.

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1 INTRODUCTION

PHENIX is a 250MWe fast breeder reactor located in Marcoule center along the Rhone river. The plant was built in the beginning of 70's, and got critical in 1973. It has been designed according to applicable seismic codes at that time.

Due to the need to continue the operation of the plant for research programs, Safety Authorities required the verification with present methods, that the essential safety functions are fulfilled for the level of earthquake defined according to the present methodologies .

The paper presents the different steps of the re-evaluation process for structures which started in 1996: general organization, methodology, general principles taken for the seismic assessment and re-evaluation, definition of the upgrades and works performed.

All the seismic upgrades are under completion and the plant is planned to re-start next year.

2 Brief description of the Plant

2.1 General

PHENIX is an experimental fast breeder reactor of pool type designed during the 60's, installed in the CEA center of MARCOULE along the Rhone river in the south part of France and is operated jointly by CEA (80%) and EDF (20%). The electric power is 250MW. The construction started in 1968 and the reactor got critical in 1973. From seismic point of view, the plant has been initially designed according to the existing codes for an earthquake level defined by the intensity level of 8 for the "Safe Shutdown Earthquake". Later on, following a modification of the operation of the plant, Safety Authority required to verify, with present methods, that the essential safety functions are fulfilled for an earthquake level defined by two response spectra. One, with a ZPA of 0.15g represents the far field sources and the second , with 0.2g, the near field.

2.2 Building description

All the buildings are on a north-south line; the ground level is 0m (figure 1). Main structures are:

The Reactor Building (RB), with main dimensions, 42m length, 26m width and 49m height, with 14m embedment (figures 5 and 6). The infrastructure, from the raft to the 8.5m elevation is a wall and floor reinforced concrete structure; common with the North Handling Building infrastructure. It supports the reactor vessel and associated systems. The superstructure is made of concrete column and steel beams for

the roof. Columns are linked by horizontally prestressed concrete plates. The columns and the plates are prestressed by vertical tendons for confinement.

The Steam Generator (SG) Building (L=42.7m, l=41m, H=43m) is founded on an independent raft with reinforced concrete wall and slab infrastructures and steel superstructures with light boarding (figure 2). It contains steam generators and secondary loops in its south part and a handling space for SG in the north.

The Turbine Building – TB - (L=52.m, l=42m, H=42m) is located in the north. It is a reinforced concrete frame, with walls and steel superstructure (figure 3). It has a turbine on a separate foundation. One row of superstructures of the SG building is founded on TB infrastructure.

The North Handling Building (NHB), which dimensions are comparable to the Reactor Building ones (figure 4), is a steel frame above the infrastructure enclosed with concrete panels with horizontal mortar slice between two panels.

Other buildings are present on the site: along the RB on the west side, auxiliary building, in the south, handling buildings. Parallel, the control room and office building and at the north east, the pump building.

All the structures are founded on a stratified alluvial soil with a slight slope to the North; the mean Young modulus is 1600Mpa.

3 Methodology of seismic reevaluation.

For this task, which is not standardized, the owner asked a Group of Experts to assist it in order to propose methodology and hypotheses of the reevaluation. FRAMATOME/NOVATOME was in charge of the general coordination and Sechaud & Metz was the Civil Engineering consultant and EDF-CLI (Engineering group in Lyon) was in charge of the control.
The different steps of the work where:

3.1 Definition of the safety function of each building and associated behavior requirements.

The earthquake is considered as an “accidental” situation and only safe shutdown, decay heat removal and limitation of radioactive release is required.

The building functions are then Capacity of Supporting safety related equipment and components (RB and NHB infrastructures, SG building, some auxiliary buildings), support of potential missiles (RB superstructure) and Stability (non collapse) for almost all the other structures.

3.2 Assessment of the “as-is” situation

This is required in order to identify the characteristics of the as-built installation, to appreciate its ageing, to check the conformity of drawings with the actual status of the plant, and to be sure of the compatibility between analysis hypotheses and the actual situation of structures. To illustrate this latter point, we may mention the detailing of concrete reinforcement in order to accept some inelastic behavior, the lap splices and the behavior of steel and concrete connections.

This task has been performed by analyzing the drawings, by walkdowns , by non destructive and destructive tests … As examples, we can mention: gaps cleaning and measurement between the different structures, friction coefficient measurement in the mortar between panels, prestressing tendons heads radiography and so on.

3.2 Seismic assessment and reevaluation general principles

Seismic assessment of an existing structure is different from the design of a new structure. In the latter situation, the design principles, based on analysis and detailing practices, induce margins in almost all the steps, due to uncertainties in the design, construction and life of the structure and to absolutely necessary simplifications in the overall process. For assessment, there is no fixed engineering practice and it is necessary to perform a critical evaluation of the analysis methods, detailing practices and criteria, in order to have an as realistic as possible situation. For this, examination of post earthquake and seismic test data was intensively used in order to help the decision making, as well as the use of conventional seismic codes such as PS92 and the Recommendations AFPS90, edited by the French Earthquake Engineering Association (AFPS). This was the main task of the Group of Experts.

Examples of critical evaluation of methods of analysis and criteria are given below.

For definition of loads, it has been suggested to consider differently far field and near field (shallow focus) earthquakes because the elastic spectrum is not a sufficient representative of the damage induced by the earthquake: duration and number of cycles play an important role. For the near field case, it has been proposed to have a reduction coefficient which modulates the elastic response.

In the load combinations, thermal loads have been excluded because they are an applied displacement which does not modify the limit load of the structure. As a complementary margin, they have been considered for the SG building.

One usual way of analysis is to have a simplified beam model for dynamic spectral analysis and, in a second step, to apply accelerations deduced from the spectral analysis where they are combined from each mode, to a 3D finite element model. This approach is not realistic and always conservative; it has been suggested to define loads in the 3D model starting from acceleration calculated by dividing forces by masses.

For structures for which only stability is required, the Group considered that some limited plastic deformation is acceptable. Conditions for this and ways to apply in verification was proposed and achieved in Turbine and Control buildings. In relation with this, some code requirements concerning the minimum longitudinal steel ratio and the lap splices was extended.

The seismic analysis is based mainly on linear models with spectral analysis on beam type models and code verification on 3D finite element models. The configuration of structures is such that “local” panels (walls or slabs) vibration modes may be excited by earthquake input signal. The Group proposed a procedure to introduce the first few local modes in the beam model in order to have a comprehensive modal behavior of structures. Limited non linear analyses with friction between panels, was used to enhance the assessment. Soil structure interaction was considered; the stratigraphy of the site make necessary to adapt the conventional impedance value, mainly for damping which is decreased compared to homogeneous soil.

4 DEFINITION OF UPGRADES

They are the result of the assessment and they can be grouped in the following manner:

Opening of joints between structures in order to avoid impact during earthquake, for RB NHB and TB.

Separation of buildings: the roof of SG building has been separated from the TB one in horizontal directions in order to avoid distortions of the steel frame.

Reinforcement or repair in order to achieve the needed resistance or to avoid fragile parts (figures 5 and 6): tying of the upper part of the RB, strengthening of lap splices with steel jackets in RB and TB,

longitudinal reinforcement in RB columns, fixing of the panels in RB and NHB, upgrading of steel structures (in NHB, SG, TB) by adding braces and improving connections, and reinforcement of anchors; in the TB, reinforcement steel has been added in order to improve the local resistance.

5 CONCLUSION

The seismic re-evaluation of PHENIX plant is a very important task which needed a dedicated organization, which includes an Expert Group to define the overall methodology, which has been successfully applied. It is based on the fact that re–evaluation of an existing structure is different from the design of a new one; all the methods and criteria must be critically examined.

The methodology was approved by Safety Authorities. Works took place essentially during 1999 and 2000 as part of an overall upgrade program. Today, they are almost completed. The overall cost of the seismic project is about 30 millions Euros.

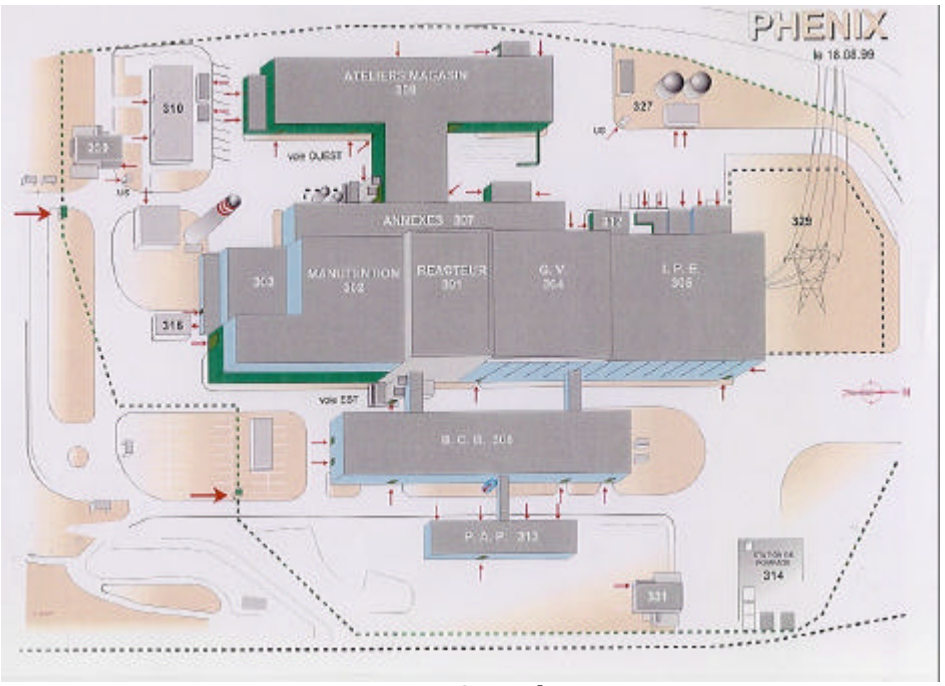

Figure 1 : General view

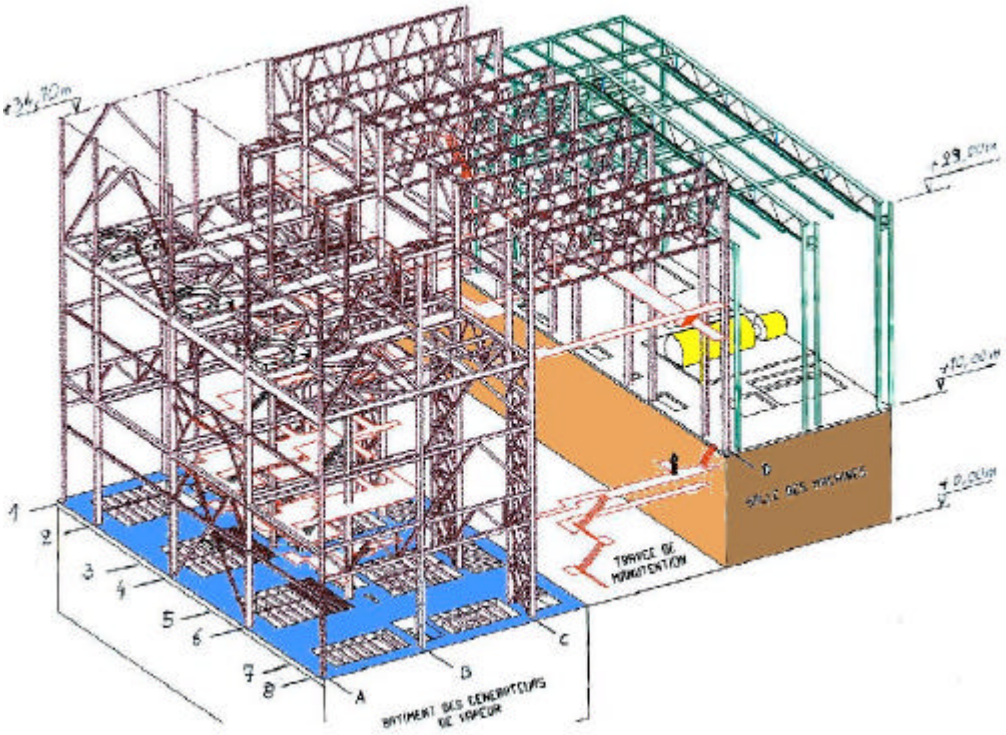

Figure 2 : Steam generator superstructure

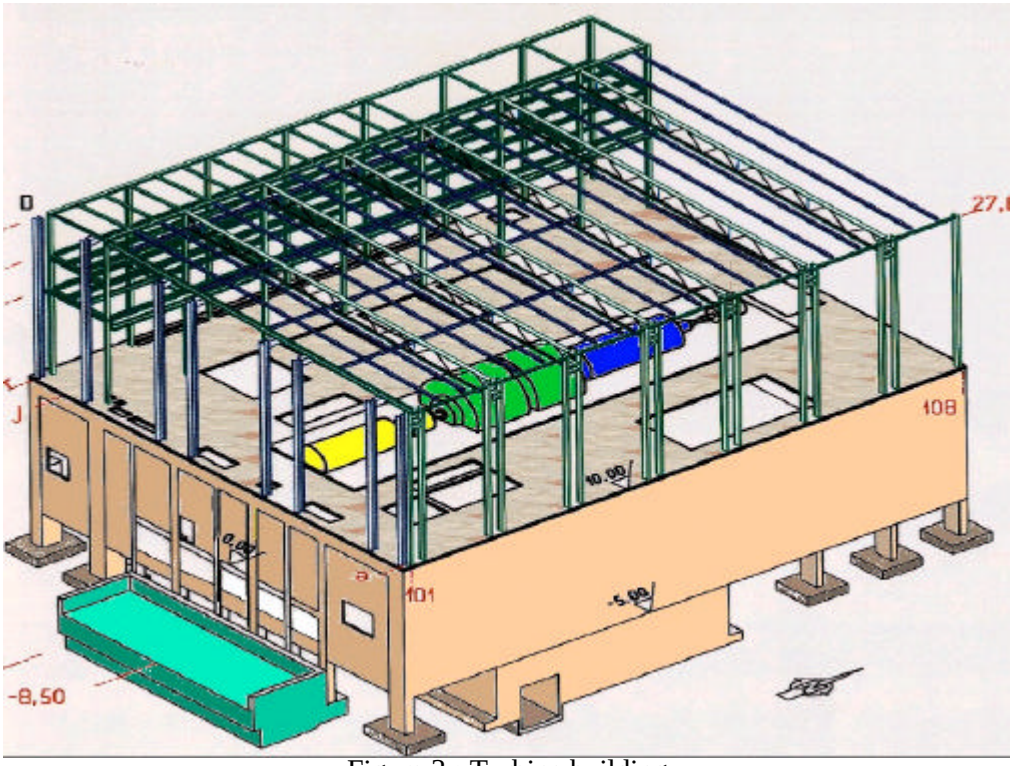

Figure 3 : Turbine building

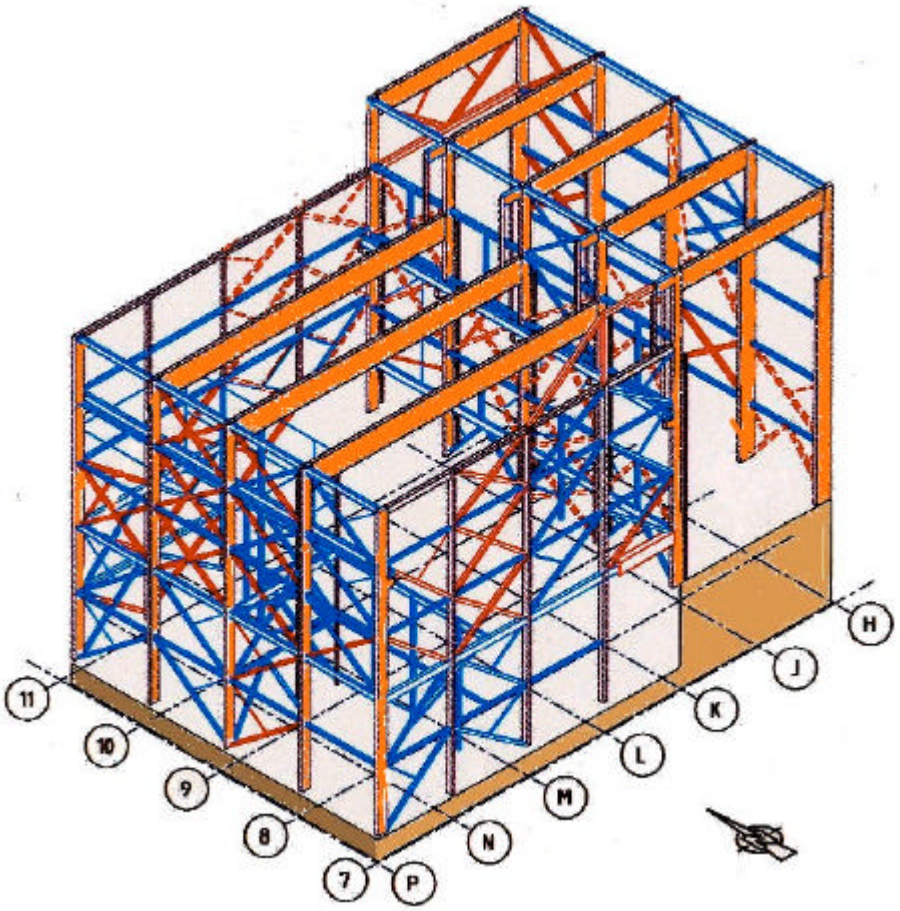

Figure 4 : North handling building superstructure

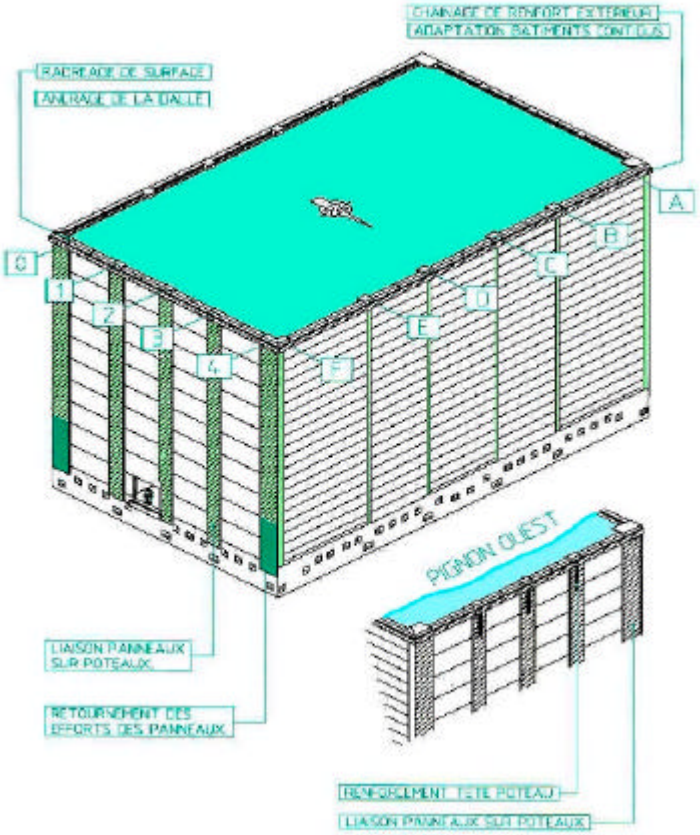

Figure 5 : Reactor building external upgrade

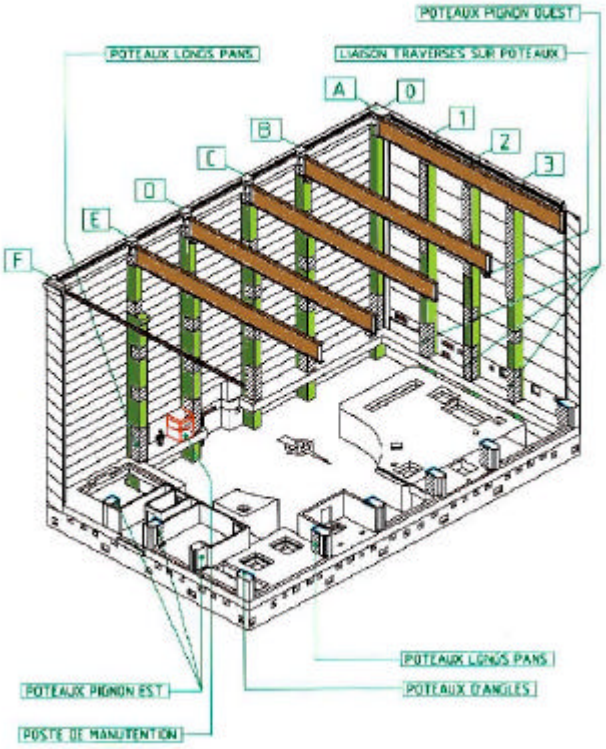

Figure 6 : Reactor building internal upgrade

D. SESSION SUMMARIES

SESSION 1: Methods and Acceptance criteria
Chairman: Mr. J.D Renard - Tractebel(B)

The following papers have been presented

- The IAEA Safety Report of seismic re-evaluation of existing NPPs by *Pierre Labbé , IAEA*
- Individual Plant Examination of External Events (IPEEE):Seismic Analyses Methods and Insights by *Charles Hofmayer, BNL (USA)*
- Seismic Re-evaluation of Kolzoduy NPP: Criteria, Methodology, Implementation by *Dr. Marin Kostov - Risk Eng Ltd, (Bulgaria)*

Intent of this introductory session was to present some of the methodologies used worldwide for seismic reassessment. Others are available or under development and some have been presented during the workshop.

The main purpose of the IAEA Safety Report under preparation is to provide guidance for conducting a seismic safety evaluation programme for an existing nuclear power plant in a manner consistent with current criteria and internationally recognized practice.
The NRC paper focuses on the seismic portion of the IPEEE analyses. It discusses acceptable methods for performing the seismic evaluation, enhancements to these methods that reflected the state-of-the-art improvements.
The last paper presents criteria and methodology used for the reassessment of Kozloduy's NPPs. It presents guidelines used and how they have been developed from those of western nuclear countries.

SESSION 1: Methods and Acceptance criteria
Chairman: Mr. K. Ohtani - NIED (Japan)

The following papers have been presented:

- Seismic Re-evaluation in the UK - A Regulators Perspective by *Mr. John Donald - HSE (UK)*
- Benefits of choosing the GIP methodology for seismic verification of equipment - The Santa Maria de Garona NPP experience by *Dora Llanos - NUCLENOR, S. A. (SP)*
- Seismic Margin Assessment of Spanish Nuclear Power Plants: a Perspective from Industry and Regulators by *Francisco Beltrán - IDOM (SP)*

SESSION 2: Countermeasures/Strengthening
Chairman: Mr. J.D. Renard - Tractebel (BE)

Four papers have been presented with different ideas which can be summarized as follows:

- A paper by Dr. Combescure about the importance of the non-linear analysis of reinforced concrete structures structures. This paper focused on differences in responses that can be observed between a classical elastic analysis using behaviour coefficients and full non-linear approaches. Dr. Combescure insisted also on the need to define correctly the accepted damage level (definition of failure, failure criterion) and on the influence of constructive details (as-built) on the ultimate behaviour of structures. Questions were asked (a) is a non-linear simulation sufficient to quantify damages to a structure when the analysis signal is different from the real one, (b) how can one take into account a second earthquake (replicas or after restart).

- The second presentation was made by Dr. Wenzel and showed how it is possible to detect damages in non easily accessible components by monitoring vibration. All information that is available by non destructive means and on in-place structures is useful in the sense that it increases safety. Monitoring methods used for non-nuclear important structures and components can also be implemented into NPPs to identify changes of signature which are evidence of some potential problems. This can also be used as a damage indicator after earthquake.

- The third paper was presented by Dr. Lee from Korea. This very interesting paper showed efforts and research of Korean authorities and utilities to create new techniques and adapt existing methods of reassessment to their specific seismic environment. Among others main information were results of work on anchorage design, efforts made to justify the CAV criterion for the Korean environment and the original and simple tool that has been developed to serve as damage indicator in case of earthquake.

- The following paper by Mr. Konno was devoted to the efforts of the Japanese nuclear authorities to improve their regulations according to lessons learned from recent earthquakes and incidents in Japan. The important information is that human factor is a big contributor to potential consequences of an earthquake. The human factor is not only the loosened capabilities of decision of operators during an earthquake, but mainly the progressive reduction of safety concern of people forgetting about the seismic threat. This raises once again the question of an automatic hardware and software protection and of a regular training of operators and safety people in charge. The second question also raised by other speakers relates to efficient housekeeping.

- The last presentation was made by R. Masopust from the Czech Republic. His paper gave an exhaustive description of tasks, methods and criteria that have been used to seismically reevaluate the components and equipments of VVER plants. Mr. Masopust showed the process used to adapt the HCLPF and GIP procedures to VVER-type equipment and components. One of the valuable

information given is the list of criteria used to identify similarities and differences between VVER's equipments and components versus those existing in the SQUG database for Western type reactors.

SESSION 2: Countermeasures/Strengthening
Chairman: Dr. T. Katona - Paks NPP (HU)

Five papers have been presented in the Session, which cover all possible aspects of the seismic re-evaluation:

- research reactors re-evaluation (HFR Petten)
- re-evaluation of prototype reactors power plants (Sollogoub)
- re-evaluation of prototype reactors power plants, regulator's point of view (Bouchon)
- methodical aspects of re-evaluation (Beliaev)
- large scale shaking table tests (Sasaki, Hattori)

The paper "Methods and practice of seismic re-evaluation for nuclear power plant structures" presented at the meeting gives some insights into the research work related to the assessment of capacity of structures and an application for the capacity evaluation of one safety related building of the Leningrad NPP. The paper shows a different practice and approach compared to usual western countries' ones. Communication between the Russian experts and experts from OECD/NEA Member States is very important for exchange of information and for obtaining a common understanding. This communication between experts is important also because the seismic re-evaluation may get more application at several Russian NPPs in the future.

The seismic re-evaluation and upgrade of Phenix reactor structures show an example of an engineering approach different from SMA and other methods developed in the USA and widely adopted worldwide. The method of re-evaluation and upgrade of the Phenix reactor building structures is based on design procedures, rational assumptions and good engineering practices. This approach could limit the amount of upgrades and lead to a rational technical solution.

Large-scale shaking table tests are extremely important for validation of design and re-evaluation techniques and to demonstrate NPP's seismic margin. Large scale shaking tables will also be the ultimate tool for confirmation of design methodologies in the future.

An essential number of research reactors have to be re-evaluated and upgraded for seismic loads. Selection of methodologies in case of research reactor re-evaluation shall follow the graded approach according to risk represented by particular reactor. IAEA guidelines and also the US DoE standards and procedures are very useful and applicable to research reactor seismic re-evaluation.

SESSION 1/2: Methods and Acceptance criteria / Countermeasures/Strengthening
Chairman: Dr. Vito Renda - JRC ISPRA (EC)

Six papers have been presented which main purpose was to summarize innovation in methodologies and tools for Seismic Hazard Assessment and/or Verification of Upgrading Measures.

- The first paper, presented by L. Pechinka, discussed the experimental verification of seismic upgrading of the coolant loop of a WWER reactor by using GERB viscous dampers. The tests proved the effectiveness of such a solution for the seismic upgrading of complex circuits.
- The second paper, presented by S. Balassanian, showed that the in-site earthquake signal should account not only for the existing seismo-tectonic and geo-technical characteristics, but also for the expected strong earthquake able to change the seismo-tectonic characteristics.
- The third paper, presented by S.K. Lee, listed recent innovation in R&D for the assessment of the seismic hazard for Korean NPP sites. R&D is focused on the improvement of earthquake catalog, seimicity, tectonic studies, ground motion and Probabilistic Safety Assessment.

- The fourth paper, presented by S. Chang, discussed the implication of Fault-Slip Rate and Earthquake Recurrence Model in the Probabilistic Seismic Hazard Assessment in the case of seismic re-evaluation of nuclear facilities.
- The fifth paper, presented by S. Vallat, showed the French operating NPP design floor response spectra, discussing in particular the relevance of soil-structure interaction and seismic re-evaluation associated margin.
- The sixth paper, presented by K. Ohtani, focused of the significant contribution of large testing facilities for Seismic Safety Assessment and Verification of Structures. In particular the new Shaking Table under construction in Japan (the largest in the world) can allow full scale testing avoiding the problems related to scaling effect.

SESSION 1/2: Methods and Acceptance criteria /Countermeasures/Strengthening
Chairman: Mr. J. Donald - HSE (UK)

Papers:

- Probabilistic seismic analysis of safety related structures of Kozloduy NPP. Dr Marin Kostov – Risk Eng Ltd (Bulgaria)

- Seismic safety re-evaluation and enhancement at Paks NPP. Dr Tamas Katona – Paks NPP (Hungaria)

- Seismic Re-evaluation program of the Armenia NPP – Results from an international co-operation project. Dr Lamberto d'Andrea – SOGIN (Italy)

- The seismic assessment of British Energy's nuclear power stations and some pragmatic solutions to seismic modifications. John MacFarlane – British Energy (UK)

- Intercomparison of analysis methods for seismically qualified isolated nuclear structures. K.N.G Fuller – TARRC (UK)

The final session was part of the countermeasures/strengthening session and considered improvements to NPP's following seismic re-evaluation. The topics included both innovative methods of improvement and more pragmatic solutions.

The NPP's which were the subject of the papers were varied including Gas Cooled Reactors, and VVER designs.

The projects which were described were in varied stages of completion. The least developed project was from Armenia where seismic hazard re-evaluations have been completed and the project is now in the process of developing the SSEL prior to the walkdowns in 2002 and any subsequent modifications. The Hungarian and Bulgarian power plants are much further along the path to seismic upgrading with all of the initial evaluations complete and many strengthening measures installed or in design. The routes by which they have got to this position varied with Kozloduy applying seismic PSA as a tool to define weaknesses and with Paks using a more deterministic approach. The results presented from Paks also included a set of procedures for post-earthquake activities.

Some of the papers detailed potential strengthening measures or other methods of providing countermeasures. The potential use of base isolation methods was described showing how they can significantly reduce design forces whilst performing in a well defined manner. Other strengthening measures described included the use of structural adhesives to significantly reduce the time and invasiveness of electrical cabinet anchorage upgrades.

CONCLUSIONS AND RECOMMENDATIONS
Chairman of the panel group: Mr. P. Sollogoub - CEA (F)

1. SEISMIC RE-EVALUATION CONCLUSIONS

1. Seismic re-evaluation is not the same as design. The allowable limits and acceptance criteria can go beyond design criteria, provided the safety is not compromised.

2. Displacement based and some non-linear analysis methods are encouraged for seismic re-evaluation.

3. Users of experience based methods, such as SQUG, should make sure the data is applicable to their specific case. Implementers of the re-evaluation should have suitable knowledge and experience. They must be trained and qualified to perform the tasks.

4. Test results and post earthquake field investigations are of great importance for seismic re-evaluation. They provide much of the information which underlies the processes used.

5. Seismic re-evaluation should make full use of as-built information and data should be verified on site as far as possible.

6. A peer review is strongly recommended as part of seismic re-evaluation. To deliver maximum benefit, it should be concurrent with the re-evaluation.

7. To improve the seismic input from reviewed earthquakes, it is necessary to fully capture the existing uncertainties and these need to be incorporated into the state of the art of the knowledge to characterize the sources, attenuation and site effects.

8. Following re-evaluation, any changes or modifications should take into account safety benefits and potential detriments.

2. GENERAL RECOMMENDATIONS ON SEISMIC DESIGN AND RE-EVALUATION

1. Human factors, housekeeping and training are important throughout the life of the facility. The seismic safety case needs to be maintained to ensure interaction hazards (eg unlocked cranes, unsecured containers,..) are not introduced between periodic walkdowns and inspections.

2. Pre-earthquake preparations need more attention, particularly the selection and recording of damage indicators which aid post earthquake decision making. A database of information collected from walkdowns (related to the condition of the plant) to compare pre and post earthquake conditions appears necessary.

3. RECOMMENDATIONS

3. There should be an improved definition of acceptance criteria for nuclear facilities which are being re-evaluated. These criteria should consider the safety consequences of acceptable damage levels, and could be risk informed rather than solely deterministic.

4. For nuclear facilities which are being re-evaluated, there should be an improved description of site specific data (input motion, geotechnical data, etc …) that reflects the knowledge and understanding of the site as appropriate. These data should be documented by an appropriate investigation program and/or monitoring system.

5. Guidelines for strengthening following re-evaluation need to be developed. These should address design and performance criteria together with practicability measures.

6. Discussion and information transfer regarding seismic re-evaluation should be pursued within the nuclear community. Cooperation with other industries with similar concerns would be of benefit.

7. The output from the seismic re-evaluation should include a review or development of the data, procedures and associated criteria which help in post earthquake decision making.

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